TECHNICAL REPORT C-69-3

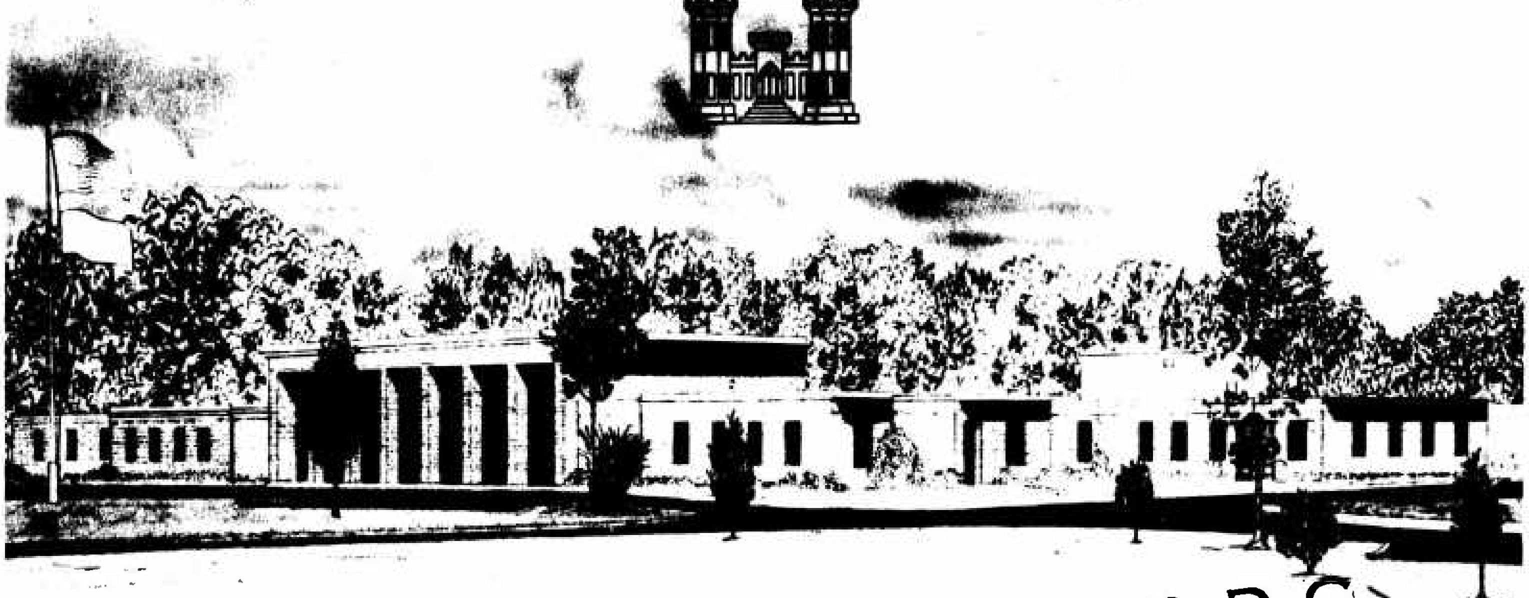
EXPEDIENT REINFORCEMENT FOR CONCRETE FOR USE IN SOUTHEAST ASIA

Report 2

PRELIMINARY TESTS OF BARBED WIRE, CONCERTINA WIRE, WIRE ROPE, LANDING MAT, AND LANDING MAT TIE BARS

by

F. B. Cox, H. G. Geymayer



March 1970

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

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ARMY-MRC VICKSBURG, MISS.

FOREWORD

A research investigation "Development and Construction Guides for Bamboo Reinforced Concrete," sponsored by the Office, Chief of Engineers (RDT&E), was authorized by program guidance, C.E.O.P.-67, RDTE Annex 2, dated 1 July 1966. The name of the program was changed to "Expedient Concrete Reinforcement" on 6 January 1967, and the program was expanded to include other potential reinforcing materials, such as barbed wire, concertina wire, AM2 landing mat tie bars, sections of M8 pierced steel landing mats, and wire rope, that are generally available near combat areas.

The work was performed during the period January 1967 to May 1969 at the Concrete Division of the U. S. Army Engineer Waterways Experiment Station under the direction of Messrs. Bryant Mather, Chief of the Concrete Division, James M. Polatty, Chief of the Engineering Mechanics Branch, Helmuth Geymayer, Chief of the Structures Section, and Frank B. Cox. This report was prepared by Messrs. Cox and Geymayer.

COL John R. Oswalt, Jr., CE, and COL Levi A. Brown, CE, were Directors of the Waterways Experiment Station during the investigation and publication of this report. Messrs. J. B. Tiffany and F. R. Brown were Technical Directors.

CONTENTS

NOTATION

a	Depth of rectangular stress block
A_R	Cross-sectional area of reinforcement
b	Beam width
C	Total compressive force carried by concrete
d	Distance from the center of gravity of reinforcement to the top fiber of concrete
E_c	Modulus of elasticity of concrete
E_R	Modulus of elasticity of reinforcement
f_c	Concrete stress
f'_c	Compressive strength of concrete
f_R	Stress of reinforcement
fy	Yield strength of reinforcement
h	Beam height
JD	Distance from centroid of compressive force to centroid of tensile force
kd	Distance from NA to the top fiber of concrete (elastic analysis)
$K_u d$	Distance from NA to the top fiber of concrete (Sinha-Ferguson and modified ultimate strength analysis)
L	Clear span length of beams
M	Internal moment of member
NA	Neutral axis of member
p	Reinforcement ratio, $p = A_R/bd$
p_b	Reinforcement ratio for balanced cross section
T	Total tensile force carried by reinforcement
W	Weight of concrete per cubic foot
ϵ_c	Strain of concrete
ϵ_R	Strain of reinforcement
ϕ	Strength capacity reduction factor

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report were converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
square inches	6.4516	square centimeters
cubic inches	16.3871	cubic centimeters
cubic feet	0.0283168	cubic meters
cubic yards	0.764555	cubic meters
pounds	0.45359237	kilograms
short tons (2000 lb)	907.185	kilograms
pounds per square inch	0.070307	kilograms (force) per square centimeter
pounds per cubic foot	16.0185	kilograms per cubic meter
inch-pounds	0.011521	meter-kilograms (force)
Fahrenheit degrees	5/9	Celsius or Kelvin degrees*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

SUMMARY

This is the second of a series of reports on expedient reinforcement for use in Southeast Asia. This report concerns materials generally available near theater of operations areas, specifically, barbed and concertina wire, wire rope, landing mat, and landing mat tie bars.

The most important engineering properties (yield strength, tensile strength, elastic modulus, and bond) of the materials were determined and 17 concrete beams were cast and tested to determine the suitability of the materials as reinforcement and to develop design procedures.

Each of the materials tested can be used as expedient reinforcement, but due to its method of fabrication, wire rope is the least desirable. Wire rope larger than 3/4 in. (1.90 cm) in diameter is not recommended as expedient reinforcement. Paint should be removed from AM2 landing mat tie bars to make them suitable as reinforcement. If it is necessary to join the tie bars to obtain a sufficient length of reinforcement, the bars should be joined by welding rather than by bolting. Barbed and concertina wire should be placed in assemblies of approximately six strands each to reduce fabrication time.

At present shear reinforcement is recommended for all types of reinforcement tested, although the shape of the M8 landing mat tested provides effective partial shear reinforcement when the sections are placed in an upright position. Either barbed or concertina wire stirrups were found to provide sufficient expedient shear reinforcement.

A modified ultimate strength (Sinha-Ferguson) method is recommended for the design of beams reinforced with barbed wire, concertina wire, wire rope, or landing mat. Either working stress or ultimate strength design according to ACI Code 318-63 is recommended for tie bar reinforcement. Shear reinforcement can be provided according to ACI Code procedures.

EXPEDIENT REINFORCEMENT FOR CONCRETE FOR USE IN SOUTHEAST ASIA

PRELIMINARY TESTS OF BARBED WIRE, CONCERTINA WIRE, WIRE ROPE, LANDING MAT, AND LANDING MAT TIE BARS

PART I: INTRODUCTION

Background

1. This is the second of a series of reports on expedient reinforcement for use in Southeast Asia, the first report having been concerned with the use of bamboo as a substitute expedient reinforcement.[1]

2. As stated in Report 1,[1] there are several areas in the world where conventional steel reinforcing bars are scarce, costly, or in some cases unavailable. Consequently, military forces, as well as civilian agencies working in these areas, have a definite interest in locally available substitute reinforcing materials that can be used as an expedient reinforcement for temporary or secondary concrete structures.

3. This second report covers materials that, although not indigenous, are generally available in and near theater of operation areas, such as barbed and concertina wire, wire rope, sections of pierced steel landing mat, and discarded landing mat tie bars (fig. 1). It appears that all of these materials are basically suitable for expedient reinforcement; however, little, if any, information is available on their most important pertinent characteristics such

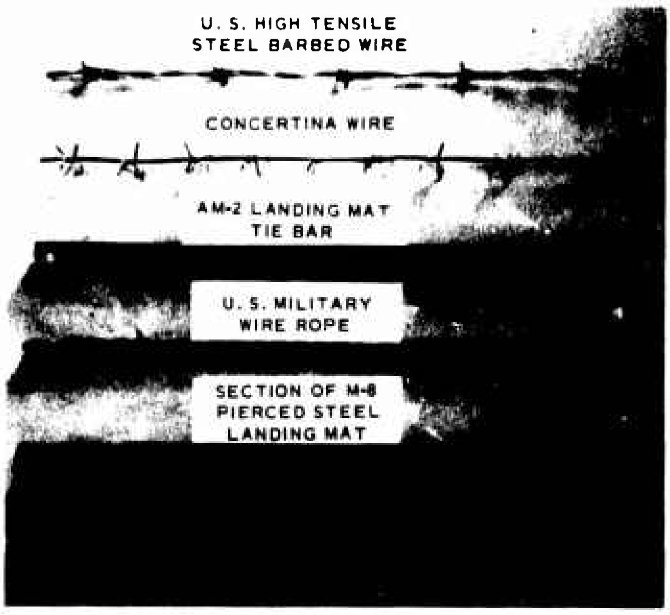


Fig. 1. Types of expedient concrete reinforcing materials investigated during this study

as elastic moduli and yield, tensile, and bond strengths, nor is there any information on how to effectively use any of these materials as an expedient reinforcement for concrete.

4. Therefore, systematic studies were undertaken to determine the essential relevant engineering properties of all mentioned materials, and methods of efficiently utilizing each or any combination of the materials as an expedient reinforcement were investigated.

Purpose and Scope

5. The purpose of this investigation is to compile and generate information concerning the use of materials such as barbed and concertina wire, wire rope, pierced steel landing mat, tie bars for landing mat, etc., as substitute reinforcement for concrete in order to form the basis for a future design and construction guide for field engineers using concrete with expedient reinforcement.

6. To accomplish these objectives, the work was divided into the following phases.

- a. Phase I. During the initial phase, the most important engineering properties of all prospective substitute reinforcing materials were determined. Tests included the determination of yield strength, tensile strength, elastic modulus, and bond with concrete. These properties were considered to be the minimum data prerequisite for an intelligent attempt to design, cast, and test structural members.
- b. Phase II. A total of 17 beams (seven reinforced with either U. S. high tensile steel barbed or concertina wire, three reinforced with wire rope, two reinforced with sections of M8 pierced steel landing mat, and five reinforced with discarded AM2 landing mat tie bars) were cast and tested during this phase to generate information on suitable reinforcing techniques.
- c. Phase III. Tentative conclusions were drawn based on the test results described in the following chapters and tentative design and analysis procedures were suggested.

7. Since this is only an interim report of a continuing investigation, it is emphasized that all conclusions and design approaches are preliminary and may be subject to revisions as the study continues and new results become available.

PART II: PROPERTIES OF SUBSTITUTE REINFORCING MATERIALS

8. Since there is little information available on the relevant engineering properties of the pertinent materials, it was deemed necessary to first establish the most important design data, such as yield strength, tensile strength, tensile modulus, and bond strength, for the potential substitute materials.

9. The following is a summary of the test procedures used, and the results obtained during this phase of the investigation.

Barbed and Concertina Wire

10. As both barbed and concertina wire have 1/2- to 1-in.* (1.27- to 2.54-cm) barbs attached to their relatively small diameters (fig. 1), bond was not considered to be a problem. Consequently, it was held sufficient to determine the yield strength, the tensile strength, and the tensile modulus of elasticity prior to an attempt to design and construct structural elements reinforced with either material.

11. Six specimens (three each of barbed and concertina wire) were prepared and tested as follows:

a. Each test specimen was cut to a length of approximately 24 in. (60.96 cm) and consisted of a complete strand of either barbed (two wrapped wires) or concertina (one crimped wire) wire.

b. An 8-in. (20.32-cm) electrical extensometer was then mounted near the middle of each specimen to measure its elongation under increasing loads (fig. 2). Stress-strain curves were plotted by an x-y recorder.

c. A uniform loading rate of 1000 psi (70.3 kg/cm^2) per minute was applied in all tests using a 30,000-lb (13,607.8-kg) universal testing machine.

12. Since, as anticipated, the stress-strain curves of neither the barbed nor the concertina wire (plate 1 and 2) showed a definite yield point, the 0.20-percent-offset method was used to determine the yield

* A table of factors for converting British units of measurement to metric units is presented on page xi.

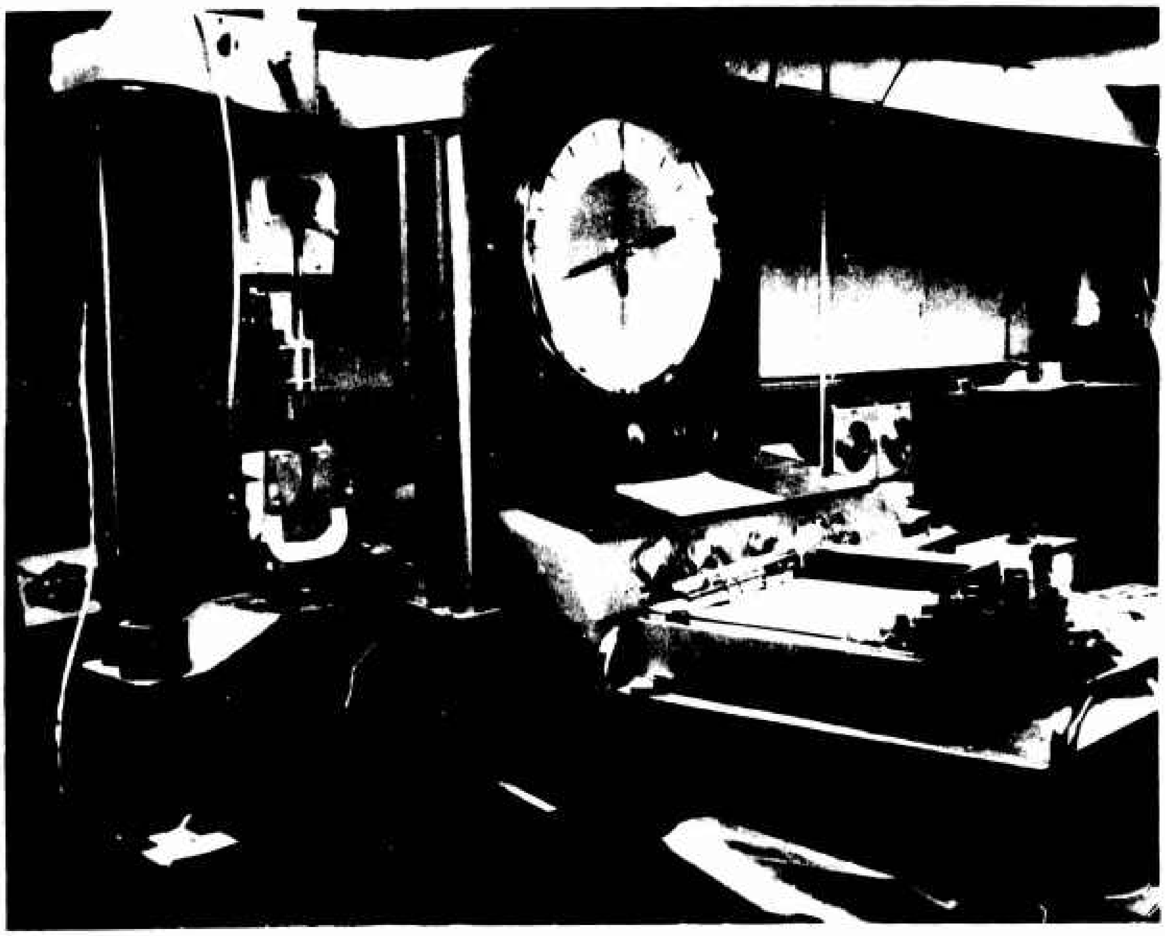


Fig. 2. Typical test arrangement used to determine the stress-strain characteristics of barbed and concertina wire

strength. The results of individual tests, listed in table 1, indicated an average yield strength of 75,000 psi (5273.0 kg/cm^2), an average ultimate tensile strength of 100,533 psi (7068.2 kg/cm^2), and an average tensile modulus of elasticity in air of 19,462,000 psi (1,368,315 kg/cm^2) for the barbed wire; and an average yield strength of 146,333 psi (10,288.2 kg/cm^2), an average ultimate tensile strength of 203,480 psi (14,306.1 kg/cm^2), and an average tensile modulus of elasticity in air of 25,920,433 psi (1,822,388 kg/cm^2) for the concertina wire. The standard deviation from the mean was within acceptable limits for all the tests; thus, the average of the three individual values was considered to be satisfactorily representative of the true material properties.

13. The yield and tensile strengths of both the barbed and concertina wire were slightly higher than expected, but their moduli of elasticity were considerably below the 29,000,000 to 30,000,000 psi (2,038,903

to 2,109,210 kg/cm^2) usually obtained for ferrous reinforcement. These relatively low values for the elastic modulus are attributed to the tendency of the barbed wire to unwrap and of the concertina wire to become uncrimped as the loads were gradually increased. Since the strands are actually restrained when embedded in concrete, it appears that, if used as reinforcement, the effective modulus of both materials may be substantially higher than indicated by the unrestrained "in-air" tension test. The effective modulus in concrete is bounded by the results of in-air tests and the 29,000,000 to 30,000,000 psi (2,038,903 to 2,109,210 kg/cm^2) normal for undeformed steel wires.

Wire Rope

14. All wire rope specimens were obtained from the Mississippi National Guard at Camp Shelby, Mississippi, and should be representative of similar wire rope used by any other particular branch of U. S. Military Forces.

15. Three particular specimens, consisting of one 1-1/4-in.- (3.18-cm-) diameter steel center "tank retriever rope," one 3/4-in.- (1.90-cm-) diameter hemp center "railway tie rope," and one 5/8-in.- (1.59-cm-) diameter hemp center "jeep wrecker cable,"* were selected from the available sources to determine their ultimate tensile strength and their modulus of elasticity. It was originally planned to also determine the yield strength of all selected specimens; however, actual testing indicated that this was impossible without inflicting damage to the extensometer, and thus yield strength data were not obtained.

16. The three selected specimens were prepared and tested as follows:

a. Each specimen was cut to a length of approximately 36 in. (91.44 cm).

b. An 8-in. (20.32-cm) electrical extensometer was then positioned near the middle of each specimen and was connected

* Sources from the Mississippi National Guard indicated that the terminology applied was representative of the primary use of each particular cable.

with an x-y recorder to obtain stress-strain curves.

c. A uniform load rate of 2550 psi (179.3 kg/cm^2) per minute was then applied to each specimen by a 440,000-lb (199,580.6-kg) universal testing machine.

d. However, as previously stated, to prevent equipment damage, the extensometer was removed from the specimen when approximately 75 percent of its estimated ultimate strength was reached.

17. The results of individual test are shown in table 2, and are summarized briefly below.

a. The modulus of elasticity of the wire ropes ranged from 6,170,000 psi (433,794 kg/cm^2) to 9,431,000 psi (663,065 kg/cm^2), which agrees rather closely with properties listed by Davis, Troxell, and Wiskocil.[2] The modulus of elasticity of the specimen containing a steel center was somewhat higher than those for specimens containing the hemp centers, as would be expected.

b. The ultimate strengths of the specimens ranged from 90,950 psi (6394 kg/cm^2) to 99,000 psi (6960 kg/cm^2) with the higher value, again, obtained on the specimen containing a steel center.

c. The stress-strain curves (plate 3) of all specimens were practically linear up to approximately 75 percent of their respective tensile strength, indicating that, due to previous use, the strands had settled around the steel or into the hemp centers, respectively, and the ropes had, therefore, lost part of their original hysteresis. Since a uniform loading rate could be achieved during the tests almost up to failure by only slight adjustment of the controls of the hydraulic testing machine, it can be concluded that the stress-strain curve of all tested ropes is fairly linear up to 90 or 95 percent of their respective tensile strengths.

Pierced Steel Landing Mat

18. Due to the cross-sectional geometry of the mats (fig. 1), which provides for substantial interlocking with the concrete, bond was not considered to be a problem; therefore, tests were limited to the determination of the yield strength, ultimate strength, and modulus of elasticity.

19. Two specimens (each acquired from panels of separate bundles) were prepared and tested as follows:

a. The specimens were oxygen cut, and tensile test coupons according to ASTM A 370 68[3] were then machined out of each specimen.

b. SR-4 strain gages of 1-in. (2.54-cm) gage length were mounted diametrically opposed near the center of each coupon.

c. A uniform loading rate of 1000 psi (70.3 kg/cm^2) per minute was applied to each specimen with a 30,000-lb (13,607.8-kg) universal testing machine.

20. Plate 4 and table 3 indicate an average yield strength for the two specimens of approximately 48,000 psi (3374.7 kg/cm^2) compared to the nominal yield strength of 35,000 psi (2460.7 kg/cm^2) quoted by the manufacturer. Much closer agreement was found between the average determined elastic modulus of 28,750,000 psi (2,021,326 kg/cm^2) and the nominal value of 29,000,000 psi (2,038,903 kg/cm^2). The average ultimate strength of the two specimens was found to be 52,965 psi (3723.8 kg/cm^2). The manufacturer did not quote the material's ultimate strength; however, it can be noted that the determined ultimate strength is approximately 10 percent higher than its tested yield strength.

Landing Mat Tie Bars

Yield strength, tensile strength, and tensile modulus

21. Six specimens were tested to determine their yield strength, ultimate tensile strength, and modulus of elasticity. The specimens were prepared and tested as follows:

a. Four specimens were tested with the original paint left on the bars, i.e., in the same condition as supplied to our field forces.

b. The paint was removed from two specimens by burning with an acetylene torch and brushing with a wire brush.

c. SR-4 strain gages of 3-in. (7.62-cm) gage lengths were mounted diametrically opposed near the center of all specimens, and stress-strain curves were obtained on an x-y recorder.

d. A uniform loading rate of 2550 psi (179.3 kg/cm^2) per minute was applied by a hydraulic test machine.

22. The results of individual tests are shown in table 4 and plate 5. Briefly, they were as follows:

a. The average stress-strain curve of all individual specimens (plate 5) indicated a definite yield point (bilinear stress-strain curve) with a yield strength of 45,390 psi (3191.2 kg/cm^2).

b. Ranges for the ultimate static tensile strength were from 66,880 psi (4702.1 kg/cm^2) to 72,870 psi (5123.3 kg/cm^2) with the average being 69,310 psi (4873.0 kg/cm^2), and the average tensile modulus was 28,983,300 psi (2,037,729 kg/cm^2) with individual values ranging from 28,000,000 psi (1,968,596 kg/cm^2) to 30,500,000 psi (2,144,364 kg/cm^2).

23. As all individual test results exhibited little variation (see table 4; maximum standard deviation from the mean is approximately 5 percent), the six tests described were considered sufficient to characterize the properties of the investigated tie bars, and although only two specimens were tested, it appeared that removing the paint by burning and brushing did not affect either the yield strength, ultimate static tensile strength, or tensile modulus of the specimens.

Bond strength

24. Fifteen AM2 landing mat tie bar specimens were tested to determine their ultimate bond strength with concrete. Method CRD-C 24-65 of the Handbook for Concrete and Cement[4] was used as a guide in preparing the bond test specimens. Figure 8 of Report 1[1] shows the test apparatus used. The concrete mixture used had a design strength of 3000 psi (210.9 kg/cm^2); however, the actual strength at the time of testing (28 days) was either 3190 psi (224.3 kg/cm^2) for batch 1 or 3260 psi (229.2 kg/cm^2) for batch 2; see table 5.

25. The specimens were prepared, cured, and tested as follows:

a. Twelve specimens were tested with the tie bars retaining their original paint.

b. The paint was removed from three specimens by burning with an acetylene torch and brushing with a wire brush.

c. All pullout specimens were moist cured in a fog room for 14 days and then room-dry cured (approximately 73 F or 22.8 C and 50 to 75 percent relative humidity) until their test date.

d. Since it was found that very little residual bond remained after initial slippage of the bar, displacement measurements at both the loaded and free ends were discontinued after initial slippage.

26. Results of individual tests, shown in table 5, indicate an average bond strength of 165 psi (11.6 kg/cm^2) for bars retaining their original paint and 388 psi (27.3 kg/cm^2) for bars with the paint removed by charring and brushing.

27. These values indicate that the bond strength between a tie bar and concrete may be increased by as much as 135 percent by simply removing the bar's initial paint. Also, the value of 388 psi (27.3 kg/cm^2) is about the bond stress one might expect for a plain reinforcing steel bar in this type of concrete, which is somewhat higher than the 250 psi (17.6 kg/cm^2) allowed for plain bars in ultimate strength design by ACI Code 318-63.[5]

PART III: BEAM TESTS

28. To date, a total of 17 beams (seven beams reinforced with either U. S. high tensile steel barbed or concertina wire, three beams reinforced with wire rope, two beams reinforced with sections of M8 pierced steel landing mat, and five beams reinforced with discarded AM2 landing mat tie bars) have been cast and tested. These and future tests are designed to determine the following.

<u>a</u>. Beams reinforced with either U. S. high tensile steel barbed or concertina wire:

(1) A practical method of placing strands of wire in the beams (such as placing individual strands or assemblages of approximately six strands each).

(2) The effect of the reinforcement ratio on the flexural capacity of the beams.

(3) The quality of bond and magnitude of allowable bond stresses.

(4) The amount of cover needed to provide an adequate corrosion protection for the reinforcement.

(5) The feasibility of using barbed or concertina wire stirrups as shear reinforcement.

(6) A suitable method for analyzing and designing concrete structural members reinforced with either barbed or concertina wire.

<u>b</u>. Beams reinforced with wire rope:

(1) The quality of bond and magnitude of allowable bond stresses.

(2) The feasibility of using other expedient reinforcement (such as barbed or concertina wire stirrups) as shear reinforcement in beams reinforced with wire rope.

(3) Whether the wire rope's rather low unrestrained modulus will result in deflections of such magnitude as to govern the design, or whether encasement in concrete improves the elastic modulus of the ropes to such an extent that deflections will not be a particular problem.

(4) Which of the available sizes and types of wire rope are most practical for use as expedient reinforcement.

(5) A suitable method for analyzing and designing concrete members reinforced with military wire rope.

c. Beams reinforced with sections of M8 pierced steel landing mat:

(1) The quality of bond and magnitude of allowable bond stresses.

(2) If upright placement of the mat sections in the beam cross section (such as shown in fig. 3, paragraph 31) will provide adequate shear strength.

(3) Whether other expedient reinforcement materials (such as barbed and concertina wire stirrups) can be used successfully as shear reinforcement if such is needed in beams reinforced with sections of pierced steel landing mats.

(4) A suitable method for analyzing or designing concrete structural members reinforced with sections of M8 pierced steel landing mat.

d. Beams reinforced with discarded AM2 landing mat tie bars:

(1) A practical method of preparing the bars (such as removing their paint) to obtain their greatest reinforcing capabilities.

(2) If expedient stirrups of barbed or concertina wire can be used effectively as shear reinforcement in concrete beams reinforced with discarded AM2 landing mat tie bars.

(3) A suitable method for analyzing and designing concrete structural members reinforced with this material.

Concrete Materials and Mixture Proportions

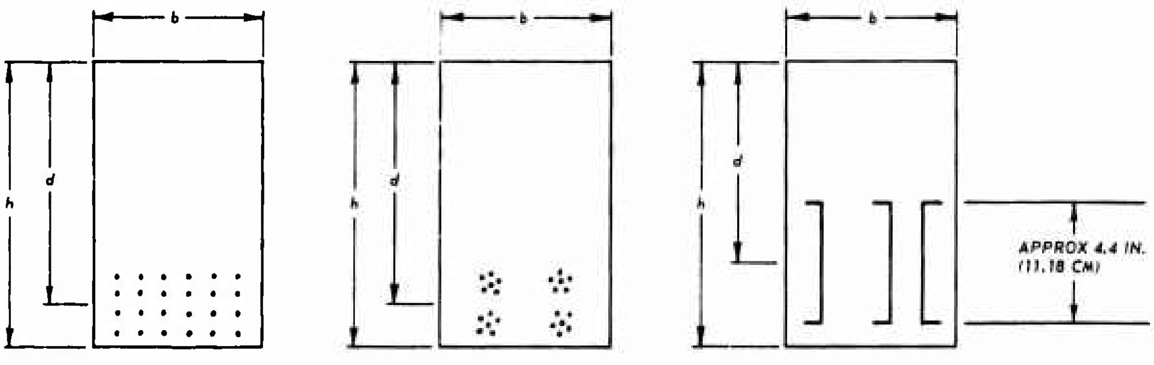
29. The materials used in the concrete mixture were Type II portland cement manufactured in Alabama and crushed limestone coarse and fine aggregate from Tennessee.

30. A concrete mixture (table 6) was proportioned with a 3/8-in. (0.95-cm) maximum size aggregate to have a slump of 2 $\pm$ 1/2 in. (5.08 $\pm$ 1.27 cm) and a 28-day compressive strength of 3000 psi (210.9 kg/cm^2). The 3/8-in. (0.95-cm) maximum aggregate size was chosen to minimize difficulties in placing and compacting the concrete where close spacing of the individual strands of barbed or concertina wire (individual strands were placed as close as 1/2 in., or 1.27 cm, on centers--OC) was required. A constant ratio of cement, aggregate, and water was maintained for all

batches of concrete. Compressive strengths of the various batches of concrete are included in tables 7-10, and 12.

Fabrication and Curing of Specimens

31. Fig. 3 shows the arrangement of the barbed wire, concertina wire, and sections of M8 pierced steel landing mat reinforcement in the rectangular beam cross sections. The AM2 landing mat tie bars and wire rope were placed in the conventional manner.



a. Individual strands of barbed or concertina wire strands 1/2 in. (1.27 cm) OC

b. Tied assemblies consisting of a maximum of six individual barbed or concertina wire strands

c. Sections of M8 pierced steel landing mat placed in a manner to increase the shear strength of a beam

Fig. 3. Arrangement of reinforcement

32. Steel forms were used in the casting of the 4- by 9- by 78-in. (10.16- by 22.86- by 198.12-cm) small beams; however, as no steel forms were available in sizes required for the large beams reinforced with either the AM2 landing mat tie bars or the 1-1/2-in.- (3.81-cm-) diameter wire rope, they were cast in plywood forms.

33. The concrete for all beams was consolidated in three layers with a 3/4-in.- (1.90-cm-) diameter head electric vibrator (frequency 7000 rpm).

34. In addition, all 4- by 9- by 78-in. (10.16- by 22.86- by 198.12-cm) beams were briefly vibrated on a vibrating table.

35. All of the beams and associated cylinders were finished with a

wooden float, stripped at a 24-hr age, and then moist cured for 13 days. After the moist curing period, the specimens were cured in laboratory air (approximately 70 $\pm$ 10 F, or 21.1 $\pm$ 5.6 C, and 50 to 80 percent relative humidity) until their test date.

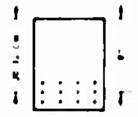
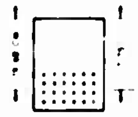
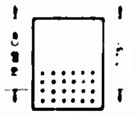
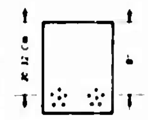
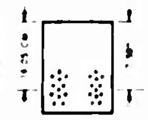
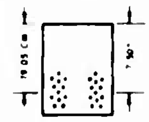
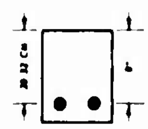
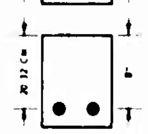
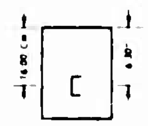
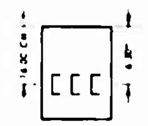
Test Methods and Results, Small Beams

36. Seven groups of simply supported beams (in all comprising seven beams reinforced with either barbed or concertina wire, two beams reinforced with wire rope, and two beams reinforced with sections of M8 pierced steel landing mats) were tested to failure under third-point loads during this test phase. Beams of groups 1 and 2 were supported on a full rocker system on one end and a half rocker system on the other end, providing a clear span of 6 ft (1.83 meters). A hydraulic system consisting of two 20-ton (18,144-kg) jacks, a control panel, and a 2500-psi (175.8-kg/cm^2) precision pressure gage, calibrated before and after the test series, was used to apply and measure the third-point loads. One-inch- (2.54-cm-) wide pads between the rollers and the beams served to distribute loads and support reactions.

37. Three dial gages independently supported, thus unaffected by possible deflection of the loading frame, were used to measure beam deflections. Total loads were applied in 500-lb (226.8-kg) to 1000-lb (453.6-kg) increments, and beam deflections were read at each load level. In all cases, the loads were removed completely at some point to check nonelastic deflections prior to continuation of loading.

38. Some small changes were made in the above procedure after testing the barbed wire and concertina wire reinforced beams. The changes consisted of: (a) substituting a half rocker system for the full rocker system, (b) substituting 3-in. (7.62-cm) dial gages for the 1-in. (2.54-cm) dial gages to allow measurement of larger deflections, and (c) using a combination of a load cell, a displacement transducer, and an x-y recorder to obtain a continuous load-midspan deflection plot.

39. The individual beams were grouped, numbered, and reinforced as follows:

Group No.	Beam No.	Type of Flexural Reinforcement	Rein-forcement Ratio $p = A_R/bd$	Type of Shear Reinforcement	Arrangement of Flexural Reinforcement
1	BW1	12 strands of U. S. high tensile steel barbed wire	0.0047	None	
	BW2	24 strands of U. S. high tensile steel barbed wire	0.0096	None	
2	BW3	12 strands of U. S. high tensile steel barbed wire	0.0047	U. S. high tensile steel barbed wire stirrups 4.0 in. (10.16 cm) OC	
	BW4	24 strands of U. S. high tensile steel barbed wire	0.0096	U. S. high tensile steel barbed wire stirrups 3.0 in. (7.62 cm) OC	
3	CW1	12 strands of concertina wire	0.0043	Concertina wire stirrups 4.0 in. (10.16 cm) OC	
4	CW2	24 strands of concertina wire	0.0089	Concertina wire stirrups 3.5 in. (8.89 cm) OC	
	BW5	24 strands of high tensile steel barbed wire	0.0099	U. S. high tensile steel barbed wire stirrups 3.0 in. (7.62 cm) OC	
5	WR1	Two 3/4-in.- (1.90-cm-) diameter wire ropes	0.0276	Concertina wire stirrups 3.0 in. (7.62 cm) OC	
	WR2	Two 5/8-in.- (1.59-cm-) diameter wire ropes	0.0191	Concertina wire stirrups 3.0 in. (7.62 cm) OC	
6	LM1*	One section of M8 pierced steel landing mats	0.0293	None	
7	LM2*	Three sections of M8 pierced steel landing mats	0.0880	None	

* Upright placement of mat sections as shown in fig. 3c appeared to substantially increase the shear strength of the beam.

40. The behavior of individual beams and the principal results of individual tests are summarized in plates 6-11 and tables 7-9. A brief discussion of group and individual beam results follows below.

Barbed or concertina wire

41. In order to achieve a 75-percent balanced cross section (p_b) according to ACI Code 318-63[5] for the barbed and concertina wire reinforced beams, approximately 30 strands of barbed wire or 32 strands of concertina wire would be required. Obviously, it is impractical to accommodate such a large number of strands in the lower portion of a 4- by 9-in. (10.16- by 22.86-cm) cross section; and from the standpoint of expedient reinforcement for temporary or secondary structures, a balanced cross section may even be uneconomical because the actual yield strength, particularly of the concertina wire, substantially exceeds the 75,000-psi (5273.0-kg/cm^2) maximum set forth by the code for reasons of crack control, which in this context appears of secondary importance. In any event, 12 and 24 strands of either barbed or concertina wire, representing a reinforcement ratio of about 28 and 56* percent (barbed wire) or 30 and 60** percent (concertina wire), respectively, of the balanced ratio p_b , were selected and are thought to be the lower and upper bounds for a practical reinforcement ratio in such beams.

42. Group 1. Results for beam BW1 are summarized below. Refer to table 7, photograph 1, and plate 6.

a. The first hairline crack was observed near midspan at a total load of 3500 lb (1587.6 kg) and cracks grew more prominent and numerous as the loads increased. At a total load of 7500 lb (3401.9 kg), the number of flexural cracks had increased greatly and some cracks had opened to 0.01 in. (0.03 cm) wide and had penetrated to a depth of approximately 7 in. (17.78 cm). Also, some flexural shear cracks† (one of which resulted in failure at a higher load) had formed between the right load and support. The mentioned flexural shear crack† developed into a complete shear crack† and caused failure at a total load of 8760 lb (3973.5 kg).

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* These values correspond to 37.5 and 75 percent, respectively, of the maximum ratio permitted by ACI 318-63.[5]

** These values correspond to 40 and 80 percent, respectively, of the maximum ratio permitted by ACI 318-63.[5]

† Terminology according to reference 6.

b. The midspan moment at the occurrence of the shear failure was 2.8 percent higher than the ultimate moment predicted by a Sinha-Ferguson[7] analysis (see Appendix A) based on the in-air stress-strain curve of the barbed wire. The calculated mean reinforcement stress of 93,300 psi (6559.6 kg/cm^2) at failure is approximately 25 percent higher than the yield strength determined by a 0.20-percent-offset method or about 93 percent of the tensile strength.

c. Although shear was the final cause of failure, failure did not occur until after the reinforcement had reached an average stress in excess of the 0.20-percent-offset yield strength; therefore, the beam had practically exhausted its flexural capacity when failing in shear. Also, the shear failure occurred at a load approximately 15 percent higher than predicted by formula 17-2 of the ACI Code[5] ($\phi = 1$).

d. A midspan deflection of approximately 1.00 in. (2.54 cm), equivalent to L/72 (where L is the clear span length of the beam), was recorded prior to the shear failure. The load versus midspan deflection curve (plate 6) of the beam indicated a relatively small and decreasing stiffness of the beam after cracking and only modest ductility.

e. The results of this beam test indicated that even concrete beams reinforced with the suggested minimum amounts of barbed wire should be provided with shear reinforcement to allow full utilization of their ultimate moment capacity.

43. Results of beam BW2 are summarized below. Refer to table 7, photograph 2, and plate 6.

a. The first hairline cracks were observed near midspan at a total load of approximately 4000 lb (1814.4 kg). At a 9000-lb (4082.3-kg) total load, about 15 flexural cracks had formed, the largest of which was 0.012 in. (0.03 cm) wide, and a flexural shear crack was noted for the first time. The cracks continued to grow more prominent until a complete shear crack of approximately 0.10-in. (0.25-cm) width and extending into a dowel crack formed between the left load and support and caused failure at a total load of approximately 11,000 lb (4989.5 kg), or under a load which was 35.3 percent higher than the predicted shear capacity of the beam according to ACI Code equation 17-2[5] ($\phi = 1$), but 16.6 percent lower than the load that would have caused the ultimate moment predicted by a Sinha-Ferguson analysis.[7]

b. A total midspan deflection of 0.75 in. (1.90 cm) was recorded preceding failure. Deflections under total loads exceeding 2500 lb (1134.0 kg) were substantially smaller than those of beam BW1 (plate 6). Due to the brittle nature of shear failures, the beam ductility was poor.

c. Increasing the reinforcement from 12 to 24 strands reduced the calculated mean reinforcement stress at failure from approximately 93,300 psi (6559.6 kg/cm^2) to approximately 64,550 psi (4538.3 kg/cm^2), or to about 85 percent of its calculated (0.20-percent-offset method) yield strength; however, by doubling the reinforcement in BW2 as compared to beam BW1, the maximum tested moment was increased from 105,120 in.-lb (1211.1 m-kg) to 132,000 in.-lb (1520.8 m-kg) or approximately 25 percent.

d. Shear reinforcement was not provided in either beam, and both beams failed essentially in shear; therefore, the results suggest strongly that shear reinforcement must be provided in beams reinforced with practical amounts of barbed wire to ensure utilization of their moment capacity and adequate ductility.

44. Group 2. The principal problem evident in the beams of group 1 was the lack of shear reinforcement; therefore, beams of the second group (BW3 and BW4) were fabricated using expedient shear reinforcement consisting of whole loops of barbed wire (fig. 4) to resist shear and diagonal tension stresses.

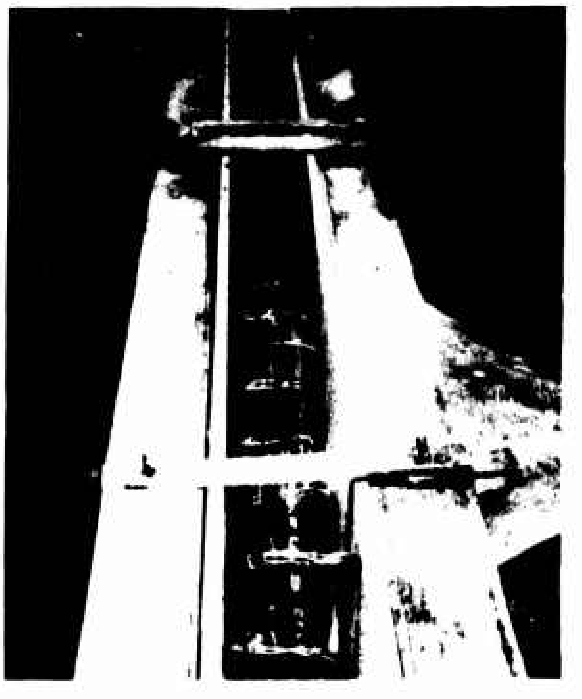


Fig. 4. Typical arrangement using either barbed or concertina wire as flexure and shear reinforcement

45. Results for beam BW3 are shown in table 7, photograph 3, and plate 7 and are briefly summarized below.

a. The first hairline cracks appeared at the same total load (3500 lb or 1587.6 kg) which caused initial cracking of beam BW1. Under a total load of 8000 lb (3628.7 kg), the number of cracks had increased to almost 20, and the length of cracks indicated a very high position for the neutral axis of the beam. A maximum crack width of 0.01 in. (0.03 cm) was measured at this point. The cracks continued to grow until a total failure load of 10,000 lb (4535.9 kg) was reached; at this load the reinforcement had strained to such

an extent that the concrete experienced a flexural compressive failure.

b. The tested ultimate moment was 10.8 percent higher than that predicted by a Sinha-Ferguson analysis.[7] A calculated reinforcement mean stress of 97,100 psi (6826.8 kg/cm^2) at failure indicates that partial yielding of reinforcement initiated the concrete's crushing in compression.

c. A midspan deflection of 1.50 in. (3.81 cm) was recorded at failure and the beam showed good ductility. The load-midspan deflection curve (plate 7) reflects somewhat smaller deflections for this beam as compared to beam BW1 at total loads in excess of 5000 lb (2268.0 kg).

d. When compared to beam BW1, the expedient shear reinforcement in the form of barbed wire stirrups in beam BW3 had increased the maximum moment from 105,120 in.-lb (1211.1 m-kg) to 120,000 in.-lb (1382.5 m-kg), or approximately 14 percent. This result confirms that shear reinforcement will increase the load-carrying capabilities of beams reinforced for flexure even with what should be considered a minimum amount of barbed wire.

46. Results for beam BW4 are shown in table 7, photograph 4, and plate 7 and are summarized below.

a. Initial hairline cracks were noted at a total load of 4500 lb (2041.2 kg). Almost 20 flexural cracks had developed at 8500-lb (3855.5-kg) total load; maximum crack width was 0.005 in. (0.01 cm). Twenty-six flexural and flexural shear cracks were noted at the 12,000-lb (5443.1-kg) load level (maximum crack width 0.01 in. or 0.03 cm); however, the flexural shear cracks did not appear to affect the flexural compressive failure at a load of 15,500 lb (7030.7 kg), corresponding to a moment 6.3 percent higher than the ultimate moment predicted by Sinha-Ferguson analysis.[7]

b. A maximum midspan deflection of approximately 1.25 in. (3.18 cm) was recorded preceding failure. A comparison of the load-midspan deflection curves for beams BW2 and BW4 (plates 6 and 7) indicates a smaller deflection for beam BW4 at all total loads exceeding 7500 lb (3401.9 kg). Due to the brittle nature of compressional failure, beam BW4 showed only modest ductility (ductility ratio about 2.0).

c. The average calculated mean stress of the reinforcement at failure was approximately 85,650 psi (6021.8 kg/cm^2), or about 14 to 15 percent greater than its 0.20-percent-offset yield strength. This suggests that the flexural compressive failure of the beam was induced by the onset of yielding of the reinforcement.

d. By comparison with beam BW2, the results show that expedient shear reinforcement increased the maximum moment from 132,000 in.-lb (1520.8 m-kg) to 186,000 in.-lb (2142.9 m-kg), or approximately 41 percent. This constitutes further proof that shear reinforcement consisting of barbed wire stirrups will definitely increase the load-carrying capabilities of concrete beams reinforced with effective amounts of barbed wire, and will help ductility.

47. Group 3. Although the test results from the two previous groups of beams were satisfactory, the method used for fabrication appeared of questionable practical value for field construction due to the excessive amount of time required for placing the reinforcement; therefore, a beam (CW1) was fabricated using 12 assembled strands of concertina wire, arranged as shown in fig. 3b, as flexural reinforcement and concertina wire stirrups as shear reinforcement.

48. Results for beam CW1 are shown in table 7, photograph 5, and plate 8, and discussed briefly below.

a. Initial hairline cracks were noted at a total load of 3000 lb (1360.8 kg), and 13 flexural cracks could be observed under a total load of 5500 lb (2494.8 kg). The maximum crack width at 5500 lb was approximately 0.012 in. (0.03 cm). At the 10,000-lb (4535.9-kg) load level, some flexural shear cracks (having a maximum width of 0.02 in. or 0.05 cm) had propagated to within 3/4 in. (1.90 cm) of the concrete's extreme compressive fibers. These cracks had almost reached the top of the beam at a load of 12,000 lb (5443.1 kg) indicating an imminent shear wedge failure. However, a sudden flexural compressive failure occurred at 13,100-lb or 5942.1-kg total load (see photograph 5e taken immediately after failure), i.e., at a midspan moment almost identical to the ultimate moment predicted by analysis.

b. A midspan deflection of 1.25 in. (3.18 cm) was recorded at failure (plate 8). Deflections at total loads exceeding 7000 lb (3175.1 kg) were larger than those of comparable beams reinforced with barbed wire. The beam showed very poor ductility, as would be expected for its mode of failure.

c. The calculated mean reinforcement stress of approximately 159,500 psi (11,214.0 kg/cm^2) was 9 to 10 percent higher than the tested 0.20-percent-offset yield strength in air. Again, as in some previous tests, the gradual yielding of the reinforcement is suspected to initiate the failure of the beam.

d. Due to the very high tensile strength of the concertina wire, beam CW1 failed at a higher moment than comparable beam BW3 reinforced with barbed wire (157,200 versus 120,000 in.-lb or 1811.1 versus 1382.5 m-kg).

e. Although other tests must be completed before a final conclusion can be reached, it appears that combining the individual strands in wire bundles of approximately six strands each is a satisfactory method, and reinforcement preparation time is reduced considerably (approximately one-half) when this method is used.

49. Group 4. Beam CW1 (group 3) indicated that the method of placing either barbed or concertina wire in assemblies as shown in fig. 3b was highly satisfactory; therefore, the two beams of this group (beams CW2, containing four six-strand assemblies of concertina wire, and BW5, containing four six-strand assemblies of barbed wire) were fabricated to further substantiate this finding.

50. Results for beam CW2 are shown in table 7, photograph 6, and plate 9, and are discussed briefly below.

a. Initial cracking occurred at a total load of 3000 lb (1360.8 kg), and the cracks became more numerous and prominent as the loads increased. Both flexural and flexural shear cracks were evident at 8000 lb (3628.7 kg) with the more prominent ones being approximately 0.005 in. (0.01 cm) in width and 6.30 in. (16.00 cm) in depth. Some of the major cracks increased from approximately 0.013 in. (0.03 cm) in width and 7.25 in. (18.42 cm) in depth at a load of 13,000 lb (5896.7 kg) (photograph 6e) to approximately 0.020 in. (0.05 cm) in width and 7.50 in. (19.05 cm) in depth at a load of 18,000 lb (8164.7 kg). Photographs 6g and 6h show the flexural compressive failure that occurred at a load of 19,700 lb (8935.8 kg).

b. A midspan deflection of 1 in. (2.54 cm), equivalent to L/72 , was recorded at the failure load. A study of the load-midspan deflection curve (plate 9) reveals that a deflection equivalent to L/360 was reached at approximately 8000 lb (3628.7 kg), or about 40 percent of the beam's maximum tested load. This indicates that deflections of similar beams will not cause particular problems when used in temporary military construction.

c. The predicted failure load for the beam was 16,500 lb (7484.3 kg); however, the actual failure occurred at a load of 19,700 lb (8935.8 kg), or about 19 percent higher than expected. As the predicted failure load was based on the

in-air stress-strain curve of concertina wire, this particular beam test supports the previously mentioned hypothesis that the unrestrained modulus may be increased considerably by concrete encasement, or in other words, that the lower portion of the stress-strain curve will be substantially different from the in-air curve if the concertina wire is embedded. It should be noted that the failure loads of all other beams reinforced with either barbed or concertina wire were predicted with reasonable accuracy; however, the computed reinforcement stress at (flexural) failure in these previous beams exceeded 80 percent of the tensile strength of the reinforcement, whereas in beam CW2 the computed reinforcement stress at failure was only about 57 percent of the tensile strength of the concertina wire. Therefore, it may be concluded that a Sinha-Ferguson[7] analysis procedure based on the in-air stress-strain curves will give conservative predictions for the ultimate moment of barbed wire and concertina wire reinforced beams, provided the beams are highly underreinforced according to ACI Code 318-63[5] (e.g. reinforcement ratio <0.7 percent for concertina wire reinforced rectangular cross sections). The reason for the inability of such an analysis to give reliable and realistic predictions at higher reinforcement ratios is probably that an in-air stress-strain curve can yield grossly unrealistic stresses for reinforcement strains smaller than about 0.008.

d. The shear reinforcement fabricated from the concertina wire was highly satisfactory.

51. Results for beam BW5 are shown in table 7, photograph 7, and plate 9, and are discussed briefly below.

a. The first visible cracks appeared at a total load of 4000 lb (1814.4 kg), or approximately 90 percent of the load required to cause initial cracking in comparable beam BW4. At a load of 6000 lb (2721.6 kg), the number of cracks had increased to 14, with the more prominent ones being approximately 0.005 in. (0.01 cm) in width and 6.00 in. (15.24 cm) in depth. Several flexural shear cracks were noted at a load of 11,000 lb (4989.5 kg), and they became much more prominent (approximately 0.01 in. or 0.03 cm in width and 6.75 in. or 17.14 cm in depth) at a load of 13,000 lb (5896.7 kg). However, as in all previous tests using barbed or concertina wire stirrups as shear reinforcement, the flexural shear cracks did not appear to affect the flexural compressive failure that occurred in this beam at a load of 14,100 lb (6395.7 kg).

b. A maximum midspan deflection of approximately 1.70 in. (4.32 cm), equivalent to L/42 , occurred at the failure load. The beam exhibited moderate ductility. A comparison

of the load-midspan deflection curves for beams BW4 and BW5 (plates 7 and 9) indicates almost equal deflection for loads up to 8000 lb (3628.7 kg); however, beam BW5 had slightly higher deflections at total loads exceeding 8000 lb (3628.7 kg).

_c_. The average calculated mean stress of the reinforcement was 82,000 psi (5765.2 kg/cm^2), or approximately 9 to 10 percent higher than its 0.20-percent-offset in-air yield strength. Again, as in beam test BW4, it appears that the flexural compressive failure was initiated by the onset yielding of the reinforcement.

_d_. Since placing the barbed wire reinforcement in assemblies resulted in a slightly smaller effective depth, the method produced beams with slightly less load-carrying capacity. However, this reduction was very small, and for reasons previously stated, bundling of reinforcement wires still appears to be the most feasible method for field construction.

52. Effects of exposure. A recent reexamination of all tested beams conducted after approximately 1 year outdoor storage subsequent to testing indicated that there was very little, if any, noticeable corrosion of either the barbed or concertina wire. And, as the beams of this investigation were provided with only a 1/2-in. (1.27-cm) cover, it appears corrosion will not be a particular problem for either the barbed or concertina wire if protective cover is provided according to section 808 of the ACI Code.[5] There is another valid reason for assuming that corrosion will not be a particular problem with barbed and concertina wire reinforcement. These materials are designed for continuous outdoor use, and can generally be expected to survive for a number of years in an outdoor environment without any protection; the additional protection afforded by the concrete cover will further reduce the probability of serious corrosion.

Wire rope, group 5

53. As previously stated, all wire rope specimens were obtained from the Mississippi National Guard at Camp Shelby, Mississippi, and should be representative of similar wire rope used by any other branch of the Military Forces.

54. A 75-percent balanced design (p_b) according to the ACI Code[5] would require a reinforcement ratio of approximately 0.0155 for both the 5/8-in. (1.59-cm) and the 3/4-in. (1.90-cm) wire rope; however, all beams

were slightly overreinforced to assure reasonably small deflections and cracks in spite of the low unrestrained elastic modulus of wire rope.

55. Results for beam WR1 are shown in table 8, photograph 8, and plate 10, and are described briefly below.

- a. Initial cracking was observed at a total load of 5000 lb (2268.0 kg), and 11 cracks were counted at 8000-lb (3628.7-kg) total load. Flexural shear cracks were noted at a load of 12,000 lb (5443.1 kg), and the more prominent cracks gradually increased from approximately 0.01 in. (0.03 cm) in width and 6.95 in. (17.65 cm) in depth at this particular load to approximately 0.02 in. (0.05 cm) in width and 7.50 in. (19.05 cm) in depth at a load of 20,000 lb (9071.9 kg). Although photograph 8e indicates that the flexural shear cracks were the most prominent at the 20,000-lb (9071.9-kg) load level, they did not appear to affect the flexural compressive failure which occurred at a load of 21,800 lb (9888.3 kg).
- b. A midspan deflection of 0.860 in. (2.18 cm), equivalent to approximately $L/84$, was recorded at the failure load. The load-midspan deflection curve (plate 10) was almost linear up to failure, i.e., the beam had practically no ductility. As expected, due to the low elastic modulus of the wire rope, the beam deflections were unusually large; however, deflections and cracking of this magnitude may be acceptable in temporary military construction.
- c. The estimated failure moment of approximately 231,210 in.-lb (2663.8 m-kg) was approximately 12 percent below the actual failure moment of 261,600 in.-lb (3013.9 m-kg). In part, this difference between actual and predicted moment capacity may again be blamed on differences between in-air and embedded strain behavior of wire rope, although for wire rope the difference is certainly not as significant as for concertina wire. However, it is conceivable that the confinement provided by the concrete may result in a somewhat higher effective elastic modulus of an embedded wire rope.
- d. Again, as in all previous tests, the shear reinforcement fabricated from the concertina wire proved to be highly satisfactory.
- e. The results of this particular test indicate that, due to the mechanism mentioned under c above, a Sinha-Ferguson analysis[7] may slightly underestimate the load-carrying capability of concrete structural elements reinforced with wire rope.

56. Results for beam WR2 are shown in table 8, photograph 9, and plate 10, and are discussed below.

a. Initial cracking occurred at a load of 3000 lb (1360.8 kg), and generally, these cracks were somewhat larger than the initial cracks of beam WR1. At a load of 5000 lb (2268.0 kg), the number of cracks had increased to six, with the larger cracks being approximately 0.02 in. (0.05 cm) in width and 7.00 in. (17.78 cm) in depth. Flexural cracks, flexural shear cracks, and short horizontal split cracks (indicating a partial loss of bond) along the layer of reinforcement were all noted at a load of 10,000 lb (4535.9 kg); however, only the flexural cracks appeared to affect the flexural compressive failure which occurred at a load of 15,900 lb (7212.1 kg).

b. A midspan deflection of 1.23 in. (3.12 cm), equivalent to L/58 , was recorded at the failure load. By comparing the load-midspan deflection curves (plate 10) of beams WR1 and WR2, it was found that beam WR2 had considerably more deflection at corresponding loads, which can be attributed to the slightly smaller reinforcement ratio of beam WR2, to the lower elastic modulus of the 5/8-in. (1.59-cm) rope as compared to the 3/4-in. (1.90-cm) rope, and perhaps to a partial loss of bond between concrete and reinforcement, indicated by the presence of split cracks.

c. The shear reinforcement, fabricated from the concertina wire, was highly satisfactory.

d. As for beam WR1, the results of this test suggest that Sinha-Ferguson analysis[7] may somewhat underestimate the load-carrying capacity of concrete structural elements reinforced with wire rope.

Landing mat, groups 6 and 7

57. All sections of M8 pierced steel landing mat that were utilized as reinforcement were oxygen cut from the individual panels and placed as shown in fig. 3c. The remaining fabrication and testing procedure was identical with that described previously for the other 4- by 9- by 78-in. (10.16- by 22.86- by 198.12-cm) beams (paragraphs 31 through 40).

58. The individual test results are summarized in table 9, photographs 10 and 11, and plate 11 and are briefly described below.

59. Beam LM1 contained one landing mat section providing a reinforcement area that resulted in a reinforcement ratio close to the $0.75p_b$ permitted by the ACI Code.[5] And, as previously stated, in an attempt to utilize the flexural reinforcement as partial shear reinforcement, the cut section of a landing mat panel was placed upright as shown in fig. 3c. Results of this test were as follows:

a. Four initial cracks appeared at a total load of 3000 lb (1360.8 kg). At 5000 lb (2268.0 kg), the number of cracks had doubled. There were approximately 10 flexural cracks visible at a load of 8000 lb (3628.7 kg) with the larger cracks being approximately 0.015 in. (0.04 cm) in width and 6.10 in. (15.49 cm) in depth. Flexural shear cracks were evident at a total load of 12,000 lb (5443.1 kg); however, they did not appear to affect the flexural compressive failure that occurred at a slightly higher total load of 12,500 lb (5669.9 kg).

b. A midspan deflection of 0.70 in. (1.78 cm), equivalent to approximately L/103 , was recorded at the failure load, indicating that deflections will not be a particular problem in beams which are provided with amounts equal to or near that specified for a balanced design (p_b) by the ACI Code.[5] In fact, due to the considerable stiffness of the mat sections in an upright position, deflections of beams reinforced with landing mat sections in the above manner will be smaller than those of conventionally reinforced beams with an equivalent reinforcement ratio.

c. The predicted failure moment (using the Sinha-Ferguson analysis[7] and an average reinforcement stress equal to the stress at the centroid of the reinforcement) was 169,940 in.-lb (1957.9 m-kg), or approximately 13 percent higher than the actual failure moment of 150,000 in.-lb (1728.2 m-kg). This indicates the probability that erroneous results will be obtained from this procedure when applied to rectangular beam sections that are reinforced with amounts near that permitted by section 1601 of the ACI Code[5] and that are not provided sufficient depths to ensure stresses in all fibers of the reinforcement equal to or near the yield strength.

d. The beam did not contain any special shear reinforcement, nominal shear stresses at failure were more than twice the allowable shear stresses according to paragraph 1701 of ACI Code 318-63,[5] and diagonal tension failure would normally have been expected at much lower loads; but shear failure was not encountered. This indicates that the upright sections of the landing mat function as partial web reinforcement, and apparently delay diagonal tension failures. This is true although the mat section did not extend up to the neutral axis of the beam and thus left the unreinforced top portion of the beam exposed to peak shear stresses.

60. Beam LM2, heavily overreinforced, was fabricated and tested to see if shear would be a problem in overreinforced beams since the results of beam test LM1 suggested that shear or diagonal tension is not a problem

in balanced or nearly balanced rectangular beam sections reinforced with sections of M8 pierced steel landing mat placed in an upright position as shown in fig. 3c. The results of this test, shown in table 9, photograph 11, and plate 11, are summarized briefly below:

a. Two very small hairline cracks appeared at a total load of 4000 lb (1814.4 kg), and the cracks became more prominent as the loads were increased. There were approximately 20 flexural cracks evident at a load of 18,000 lb (8164.7 kg); however, these cracks were relatively small, the most prominent being about 4.50 in. (1.14 cm) in depth and 0.005 in. (0.01 cm) in width. A flexural compressive failure occurred at a load of 19,350 lb (8777.0 kg).

b. A midspan deflection of 0.49 in. (1.24 cm), equivalent to approximately L/147 , was recorded at the failure load, again demonstrating the stiffness of beams containing landing mat reinforcement arranged as shown in fig. 3c. Also, as normal in overreinforced beams, ductility was poor.

c. The predicted failure load was 17,994 lb (8161.9 kg), but the actual failure load was 19,350 lb (8770.0 kg), or approximately 7.5 percent higher than expected. Again, it should be emphasized that the average reinforcement stress was taken as the stress at the centroid of the steel, which is permissible if the reinforcement has yielded throughout or if the reinforcement's stresses are well within the linear section of its stress-strain curve (which was the case for beam LM2). (In the latter case, the increase in moment capacity due to the flexural stiffness of the reinforcement itself should be considered.) However, if neither of the above prerequisites is fulfilled, this method will give erroneous results; therefore, checks should be made to see if either of the two above conditions exists before using the centroidal stress as the average reinforcement stress.

d. Again, as in beam test LM1, shear or diagonal tension did not appear to be a problem for beams of this type when the reinforcement extends to a height of approximately h/2 .

Test Methods and Results, Large Beams

61. Four groups of simply supported beams, involving a total of five 5- by 12- by 138-in. (12.70- by 30.48- by 350.52-cm) beams reinforced with discarded AM2 landing mat tie bars and one 7- by 15- by 180-in. (17.78- by 38.10- by 457.20-cm) beam reinforced with a 1.25-in.- (3.18-cm-) diameter

steel-center wire rope, were tested to failure during this phase. Tests were also conducted to determine a suitable method for splinting the AM2 landing mat tie bars.

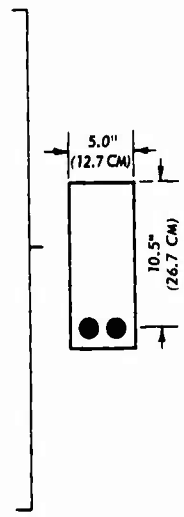
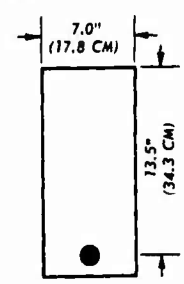
62. The beams reinforced with discarded AM2 landing mat tie bars or with the 1.25-in. (3.18-cm) wire rope were provided with clear spans of 11.00 ft (3.35 m) and 14.50 ft (4.42 m), respectively, and supported on one end with a half rocker system. All beams, except beam AM2-4 (whose bond-development length was increased by loading at its midspan only), were tested under third-point loading using a single hydraulic jack and a wide flange distributing beam to apply the third-point loads. All loads were measured by a calibrated mechanical load cell, and both loads and support reactions were transmitted to the beam through steel pads of 1-in. (2.54-cm) widths.

63. The loads were applied in 500- to 1000-lb (226.8- to 453.6-kg) increments and, as in previous tests, were removed at some point to check the nonelastic deflection of each beam. Three dial gages of 3-in. (7.62-cm) travel lengths, mounted so as to be unaffected by deformation of the beam supports, were used to measure the beam's deflection under each load increment.

64. All beams reinforced with AM2 landing mat tie bars contained two bars (almost a balanced section p_b according to ACI Code 318-63[5]) with the bar ends extending beyond the ends of the beams so that bond slippage could be monitored by dial gages (fig. 5). No bond slippage measurements were made on beam WR3, which was reinforced with one 1.25-in.-(3.18-cm-) diameter wire rope. This beam, also, was very nearly balanced.

Fig. 5. Typical placement of dial gages used in monitoring slippage between concrete and reinforcement

65. The individual beams were grouped, numbered, and reinforced as follows:

Group No.	Beam No.	Type of Flexural Reinforcement	Reinforcement Ratio $p = A_R/bd$	Type of Shear Reinforcement	Arrangement of Reinforcement
1	AM2-1	Two AM2 landing mat tie bars retaining their original paint	0.0299	None	
	AM2-2	Two AM2 landing mat tie bars retaining their original paint	0.0299	None	
2	AM2-3	Two AM2 landing mat tie bars with their original paint removed	0.0299	None	
	AM2-4	Two AM2 landing mat tie bars with their original paint removed	0.0299	None	
3	AM2-5	Two AM2 landing mat tie bars with their original paint removed	0.0299	U. S. high tensile barbed wire stirrups 2.25 in. (5.72 cm) OC	
4	WR-3	One 1-1/4-in.- (3.18-cm-) diameter wire rope	0.0129	U. S. high tensile barbed wire stirrups 3.5 in. (8.89 cm) OC	

66. Their behavior and the principal results of individual tests are summarized in plates 12, 13, 14, and 16 and tables 10 and 12. Also, they are described briefly below.

<u>Group 1</u>

67. As the tie bars were supplied with a painted surface, it was decided to first determine their reinforcing capabilities as supplied; therefore, group 1 consisted of two beams (AM2-1 and AM2-2) reinforced with bars retaining their original paint.

68. Test results for beam AM2-1 are shown in table 10, photograph 12, and plate 12, and are discussed below.

<u>a</u>. The first hairline cracks were noted at a total load of 3500 lb (1587.6 kg) with the cracks becoming more prominent as the loads increased. At a total load of 8500 lb (3855.5 kg) a flexural shear crack of approximately

0.02-in. (0.05-cm) width and 8-in. (20.32-cm) depth had formed under the right loading point. The dial gages monitoring bond slippage between the concrete and reinforcement indicated that the first measurable slippage occurred near a total load of 5000 lb (2268.0 kg) and that the bond between the concrete and reinforcement was completely destroyed at a total load of 10,500 lb (4762.7 kg).

b. A midspan deflection of 0.633 in. (1.61 cm) was recorded at the failure load, representing a deflection ratio of approximately L/210 .

c. The calculated mean reinforcement stresses (table 10) of 16,880 psi (1186.8 kg/cm^2) using an elastic analysis and 15,450 psi (1086.2 kg/cm^2) using a modified ultimate strength analysis (based on slippage of the reinforcement at a constant load; see Appendix B) indicated that only a small portion (approximately 35 percent) of the reinforcement's yield strength was utilized.

d. The tested ultimate moment of 231,000 in.-lb (2661.4 m-kg) (due to bond failure) compared to a predicted moment at shear failure of 366,600 in.-lb (4223.6 m-kg) using formula 17-2 of ACI Code 318-63[5] for beams not reinforced for shear indicates a premature bond failure.

e. Results listed under c and d and a calculated bond strength of 96 psi (6.7 kg/cm^2) using a working stress analysis or 103 psi (7.2 kg/cm^2) using a modified ultimate strength analysis clearly indicate that the bond strength of the painted tie bar must be improved before it can be used effectively as reinforcement.

69. Test results for beam AM2-2 are shown in table 10, photograph 13, and plate 12, and are discussed below.

a. Hairline cracks first appeared near the left loading point at a total load of 3000 lb (1360.8 kg), with the next significant change being noted at 5500 lb (2494.8 kg). There were approximately five flexural cracks visible at a load of 9000 lb (4082.3 kg), the largest being approximately 0.015 in. (0.04 cm) in width and 7 in. (17.78 cm) in depth. The first measurable slippage did not occur until a load of approximately 9500 lb (4309.1 kg) was reached; however, as in beam AM2-1, the dial gages indicated a complete loss of bond at a load of 10,500 lb (4762.7 kg).

b. A midspan deflection of 0.423 in. (1.07 cm), or approximately 70 percent of the deflection for beam AM2-1, was recorded at the failure load; however, the two beams' load-midspan deflection curves (plate 12) reveal deflections of near equal magnitudes for all other corresponding loads. Both beams showed poor ductility.

c. Other results were similar to those reported for beam AM2-1, and further substantiate the conclusion that an improvement in the painted bar's bond strength is necessary before it can be used effectively as reinforcement.

Group 2

70. As the results of tests conducted on the beams of group 1 indicated a premature failure due to a loss of bond, an attempt was made in this second group to improve the situation by removing the original paint from the tie bars with the aid of an acetylene torch and a wire brush.

71. As previously stated, beam AM2-3 was tested under third-point loading; however, to obtain a greater bond-development length for the reinforcement, beam AM2-4 was tested by applying the load at the midspan only.

72. Test results for beam AM2-3 are shown in table 10, photograph 14, and plate 13, and are discussed below.

a. The first hairline cracks appeared at a total load of 6500 lb (2948.4 kg), and 11 flexural cracks were noted at 9500 lb (4309.1 kg). At 13,500 lb (6123.5 kg), the number of cracks had increased to 13, with the larger cracks being approximately 6 in. (15.24 cm) in depth and 0.01 in. (0.03 cm) in width. A complete shear crack, together with an accompanying dowel crack of approximately 2-in. (5.08-cm) width, caused sudden end anchorage failure at 16,000 lb (7257.5 kg).

b. A comparison of the load-midspan deflection curves for beams AM2-1, AM2-2, and AM2-3 (plates 12 and 13) shows considerably smaller deflections for beam AM2-3 under loads exceeding 1500 lb (680.4 kg).

c. The tested maximum moment of 352,000 in.-lb (4055.4 m-kg) for beam AM2-3 compared to the 231,000 in.-lb (2661.4 m-kg) obtained for beams AM2-1 and AM2-2 indicated a moment increase of approximately 50 percent due to removing the original paint. The lack of shear reinforcement in beam AM2-3 appeared to have caused premature failure in spite of the fact that the shear stresses at failure (approximately 150 psi or 10.5 kg/cm^2) were slightly below the shear stresses (approximately 160 psi or 11.2 kg/cm^2) permitted by ACI Code equation 17-2.[5]

d. The calculated mean stress of 25,650 psi (1803.4 kg/cm^2) using an elastic analysis or 24,600 psi (1729.6 kg/cm^2) using a modified ultimate strength analysis (table 10) is approximately 55 percent of the tie bar's calculated yield

strength. Again, this shows a definite improvement over results on painted bars.

e. The results listed under a, b, c, and d, and an improvement in bond strength of about 52 percent (working stress analysis) to 60 percent (modified ultimate strength analysis) all lead to the conclusion that an increase in flexural strength of at least 50 percent may be obtained by simply removing the paint from the bar's surface. An even higher increase should be possible through the concurrent use of shear reinforcement.

73. As previously stated, beam AM2-4 was loaded at the midspan only. Results of this test, shown in table 10, photograph 15, and plate 13, were as follows:

a. The first hairline cracks appeared directly under the loading point at a total load of 5500 lb (2494.8 kg). At 11,000 lb (4989.5 kg), a total of nine cracks had developed, with the larger cracks being approximately 5 in. (12.70 cm) in depth and 0.01 in. (0.03 cm) in width. At 16,000 lb (7257.5 kg), some flexure shear cracks were noted. Failure occurred at a total load of 16,250 lb (7370.9 kg) as a result of the sudden development of a number of horizontal cracks, which appeared to be a combination of tensile shear, split, and dowel cracks (photograph 15d, taken immediately after failure). The failure may be classified as essentially a shear failure and could have been prevented by shear reinforcement. Beam ductility was rather poor.

b. Due to the midspan loading, the measured beam deflections (plate 13) were larger than those of beam AM2-3 under equivalent third-point loads.

c. The maximum moment tested at 536,250 in.-lb (6178.1 m-kg) and calculated mean reinforcement stress of 39,080 psi (2747.6 kg/cm^2) using an elastic analysis, or 42,220 psi (2968.4 kg/cm^2) using a modified ultimate strength analysis, indicate a substantial increase over beam AM2-3 quantities; however, this can be attributed only to the different loading arrangement for the two beams which resulted in higher ultimate moments and reinforcement stresses in beam AM2-4, although both beams failed under almost exactly the same shear load.

d. An average bond stress of approximately 148 psi (10.4 kg/cm^2) using a working stress analysis, or 188 psi (13.2 kg/cm^2) using a modified ultimate strength analysis, was calculated for the failure load.

Group 3

74. As shear contributed to the failure of beams AM2-3 and AM2-4, it was decided to test a similar beam containing properly designed shear reinforcement. Therefore, group 3 consisted of only one beam (beam AM2-5). Due to the probable unavailability of standard shear reinforcement (stirrups) in the field, expedient barbed wire ties (stirrups) were fabricated and used as the shear reinforcement (fig. 6). Results of this test, shown in table 10, photograph 16, and plate 14, were as follows:

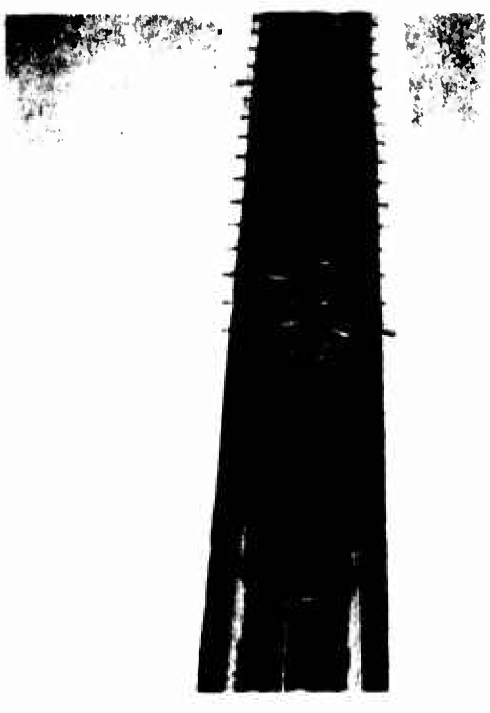


Fig. 6. Typical arrangement using landing mat tie bars and either barbed or concertina wire stirrups as flexure and shear reinforcement

a. The first hairline cracks appeared at 4000 lb (1814.4 kg). Some flexural shear cracks were evident at 15,000 lb (6803.9 kg), with the larger cracks being approximately 7 in. (17.78 cm) deep and 0.01 in. (0.03 cm) wide. At 22,000 lb (9979.0 kg), the cracks had increased in both size and number, and the gages monitoring slippage showed some loss of bond. A flexural bond failure occurred very suddenly at a load of 22,800 lb (10,341.9 kg).

b. A midspan deflection of 1.22 in. (3.10 cm), equivalent to approximately L/108 . was recorded at the failure load; however, the load-midspan deflection curves (plates 13 and 14) for beams AM2-3 and AM2-5 (comparable beams except beam AM2-3 contained no shear reinforcement) indicate similar deflections for both beams at corresponding loads.

c. Comparing the maximum tested moments of 352,000 in.-lb (4055.4 m-kg) for beam AM2-3 and 501,600 in.-lb (5778.9 m-kg) for beam AM2-5, it can be concluded that the shear reinforcement increased the load-carrying capacity by approximately 42.5 percent.

d. Also, when comparing beams AM2-3 and AM2-5 (table 10), the calculated mean stress of the reinforcement was increased from approximately 25,650 psi (1803.4 kg/cm^2) using an

elastic analysis or 24,600 psi (1729.6 kg/cm^2) using a modified ultimate strength analysis to 36,260 psi (2549.3 kg/cm^2) using the same elastic analysis, or 35,790 psi (2516.3 kg/cm^2) using the same modified ultimate strength analysis. This represents an increase of approximately 40 to 45 percent in the stress of the reinforcement, resulting in the utilization of about 80 percent of the tie bar's yield strength prior to bond failure.

e. Therefore, from the above-mentioned results it can again be concluded that barbed wire stirrups are an effective expedient shear reinforcement.

Landing mat tie bar splints

75. Although the length of the AM2 landing mat tie bars (144 in. or 365.76 cm) was sufficient for the beams used in this investigation, it may be necessary in some cases to splint several bars together to provide a sufficient length of reinforcement; therefore, two possible methods of splinting the bars, welding and bolting, were investigated and evaluated during this phase of the investigation. The specimens were fabricated, tested, and evaluated as follows.

76. Welded splints. All welded specimens were electric-arc butt-welded utilizing a 60-deg double-vee joint with an angle backup that had a 1/8-in. (0.32-cm) gap between the ends of the specimens as shown in fig. 7. (The angle backup was used to minimize warping and misalignment of the specimen during the welding operation.)

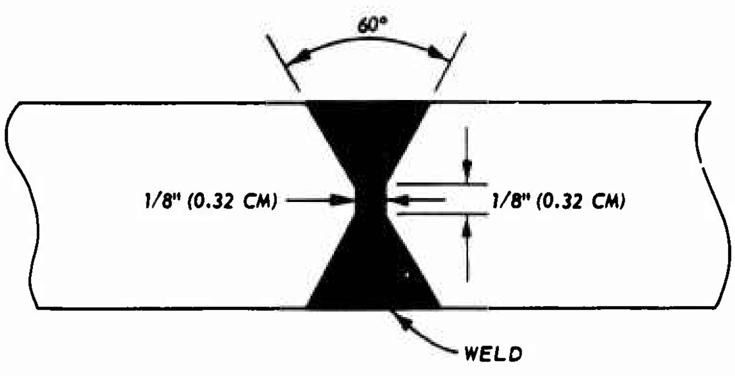


Fig. 7. Method of welding AM2 landing mat tie bars

77. Also, all specimens were welded at room temperature (no preheat or postheat was applied to any specimen) using Fleet Weld No. 35 welding electrodes of 5/32-in. (0.40-cm) diameter, which had been stored in a temperature and humidity controlled cabinet prior to use. The welding was performed by a certified welder using a 300-amp, 40-volt welder at 130 amp of direct current.

78. Five welded specimens were tested for either their static tensile or bending capabilities. The tensile test procedure was as follows:

a. Each test specimen was cut to a length of approximately 36 in. (91.44 cm).

b. An 8-in. (20.32-cm) electrical extensometer was positioned near the center of each specimen (center of specimen was also center of weld) to measure its elongation under increasing loads, and a stress-strain curve was plotted by an x-y recorder.

c. A uniform loading rate of 2550 psi (179.3 kg/cm^2) per minute was applied by a hydraulic testing machine.

79. The results of the tensile tests are shown in table 11 and plate 15, and are summarized briefly below.

a. The average yield strength of the three welded specimens was 42,670 psi (3000.0 kg/cm^2) or approximately the same as the 43,000 psi (3023.2 kg/cm^2) found for the yield strength of the control specimen.*

b. An elastic modulus of approximately 29,000,000 psi (2,038,903 kg/cm^2) was obtained for both the control and welded specimens.

c. All welded specimens failed within the weld, resulting in a decrease in tensile strength of approximately 20 percent.

d. The average elongation was 3.62 percent, i.e., considerably less than the 10 percent minimum specified by section A 615-68 of the ASTM Standards[3] for deformed billett-steel concrete reinforcement. As the control bar elongated approximately 9 percent, the reduction in elongation can be attributed to the reduced tensile strength and ductility of the weld.

80. Two bond test specimens were tested according to section A 615-68 of the ASTM Standards[3] (fig. 8). Both specimens cracked on the outside bent portion of the weld; however, the cracks were very small and could be observed only under magnification.

81. Bolted splints. Each bolted splint was fabricated by placing 0.50-in.- (1.27-cm-) diameter high tensile steel bolts (manufacturer's information indicated that the bolt's yield strength was greater than 60,000 psi or 4218.4 kg/cm^2) through the 0.50-in.- (1.27-cm-) diameter eyelets which are located at one end of each bar (fig. 9).

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* A control test was run on an unwelded portion of the same tie bar specimen.

Fig. 8. Test arrangement used to determine the bending properties of AM2 landing mat tie bar welded splints

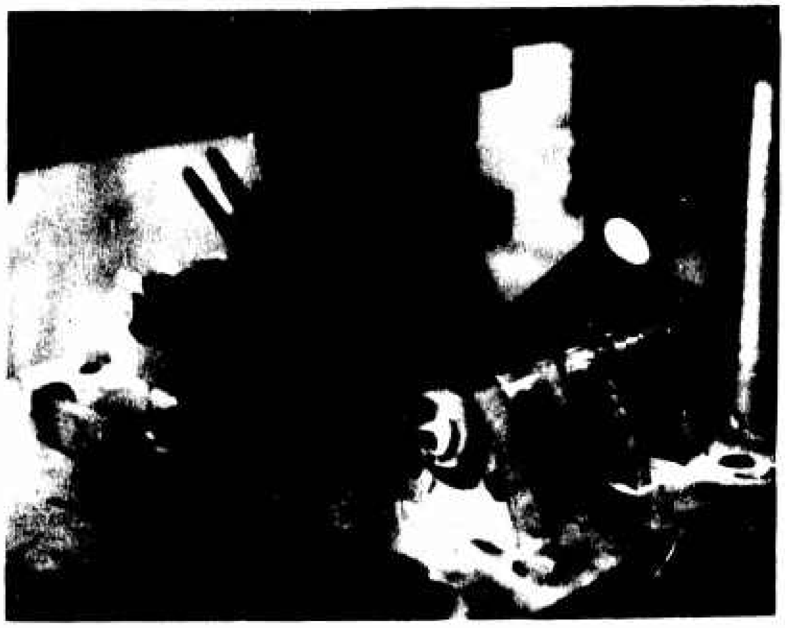

Fig. 9. Method used to fabricate AM2 landing mat tie bar bolted splints

82. Two bolted specimens were tested in tension by the same procedure described previously for the welded tensile tests, except that no strain measurements could be made because the extensometer could not be placed over the bolted section.

83. The specimens failed by shearing the bolts at loads of 12,900 lb (5851.3 kg) and 13,000 lb (5896.7 kg), respectively. Since these loads are only approximately 19 percent of the tensile strength of the bar, the method must be considered unsatisfactory; therefore, no additional tests were conducted on bolted splints.

84. Conclusions. On the basis of the above-described test results, the following conclusions were drawn on AM2 landing mat tie bar splints.

a. Although the welded specimens did not meet the specified criteria, welded splints can probably be used satisfactorily

in temporary military construction, if allowable stresses (or the design yield strength) for the splices are reduced by approximately 20 percent.

b. Without modification, bolted splints should not be used because of the probability of shearing the connecting bolts (even high strength bolts) at loads of approximately 19 percent of the tensile strength of the AM2 landing mat tie bar. Modification by slightly increasing the diameter of the eyelet and then using a slightly larger bolt may correspondingly increase the tensile load. However, this is not considered to be a feasible field operation; therefore, as previously stated, only welded splints are recommended at this time.

Group 4

85. Due to the size (1.25-in. or 3.18-cm diameter) and tensile strength of the particular type of wire rope used for beam WR3, a somewhat larger beam cross section of 7 by 15 in. (17.78 by 38.10 cm) was required for a 75-percent balanced design (p_b , ACI Code 318-63[5]), and as for most previous beams, shear reinforcement was provided in the form of concertina wire stirrups. Results of this test, shown in table 12, photograph 17, and plate 16, were as follows:

a. Initial cracking occurred at a total load of 2000 lb (907.2 kg) with the largest crack being approximately 0.06 in. (0.15 cm) wide and 12.25 in. (31.12 cm) deep. The next significant change (especially in the number of cracks) was noted at a total load of 6000 lb (2721.6 kg) and some of the previously mentioned cracks had grown to about 0.08 in. (0.20 cm) wide and 13.52 in. (34.34 cm) deep. The cracks continued to increase in both size and number, until a sudden failure (probably due to loss of bond between the concrete and reinforcement) occurred at a load of 10,000 lb (4535.9 kg).

b. Due to the low modulus of the wire rope, beam deflections were large, although they may still be considered acceptable for temporary military construction; however, the low failure load was disappointing. A midspan deflection of 1.50 in. (3.81 cm), equivalent to approximately L/116 , was recorded prior to sudden failure.

c. The predicted load-carrying capacity was 32,280 lb (14,642.0 kg) (excluding bond failure) but actual failure occurred at the previously mentioned load of 10,000 lb (4535.9 kg); therefore, it is evident that only a small portion of the wire rope's strength was used effectively. For

this reason, and in view of the fact that its ultimate moment was only 3.25 times greater than the moment capacity one would expect for a similar unreinforced section (using 1/10 the compressive strength as the tensile strength of the concrete), the beam performance must be considered unsatisfactory.

d. As the tests conducted on beams WR1 and WR2, which contained reinforcement consisting of 3/4-in. (1.90-cm) and 5/8-in. (1.59-cm) wire ropes, respectively, had yielded satisfactory results, it may be concluded that military wire ropes up to perhaps 3/4-in. (1.90-cm) diameters can be used as expedient concrete reinforcement, but ropes with larger diameters should be avoided unless special methods are used to provide end anchorage. The main problems associated with the use of wire rope as expedient reinforcement are its low modulus and the possibility of insufficient bond strength. Both modulus and bond strength vary substantially with rope geometry, type, and size, and are furthermore dependent on the rope's previous use. Obviously for bond the protrusion to diameter ratio of the rope and the effectiveness of grease removal methods will be additional important factors.

86. Based on these considerations and on the results of the three beam tests described, it may be tentatively concluded that only ropes with diameters not in excess of 3/4 in. (1.90 cm) should be used as expedient reinforcement, that the ropes should be carefully treated to remove grease and oil, and that high reinforcement ratios must be utilized to ensure reasonably small deflections and cracks.

PART IV: CONCLUSIONS

87. On the basis of the test results described in the previous parts, the following conclusions can be drawn. It should be emphasized, however, that they are tentative and may be subject to revision as additional test results become available.

Engineering Properties of Expedient Reinforcing Materials

Barbed and concertina wire

88. Both the investigated barbed and the concertina wire possess high yield (0.20-percent-offset method) and tensile strength; however, due to the barbed wire being wrapped (one complete strand consists of two individual wires) and the concertina wire being crimped (one single crimped wire) their tensile moduli in air are substantially lower than those usually obtained for ferrous reinforcement. The following are the average values found for their yield strength, tensile strength, and unrestrained elastic modulus during this investigation.

Material	Yield Strength	Tensile Strength	Tensile Modulus in Air
Barbed wire	75,000 psi ($5273.0\ kg/cm^2$)	100,533 psi ($7068.2\ kg/cm^2$)	19,462,000 psi ($1,368,315\ kg/cm^2$)
Concertina wire	146,333 psi ($10,288.2\ kg/cm^2$)	203,480 psi ($14,306.1\ kg/cm^2$)	25,920,433 psi ($1,822,388\ kg/cm^2$)

89. As anticipated, bond proved to be no problem in concrete structural elements reinforced with either barbed or concertina wire due to the small diameter of both wires, the wire deformations, and the additional barbs.

90. Since WES correspondence with local barbed wire distributors indicates that there are more than 500 different types of barbed wire manufactured throughout the world, it is impossible to draw conclusions concerning the general use of barbed wire as expedient reinforcement. Barbed wire of the type investigated in this study (U. S. high tensile steel barbed wire), or barbed wire with similar properties, certainly is a

suitable expedient reinforcing material, but some other types of barbed wire, such as those consisting of a crimped single-strand low-strength wire, are equally certain to give unsatisfactory results.

91. The conclusion reached with regard to concertina wire appears to be more generally valid, since concertina wire is used almost exclusively by military forces and is manufactured to their specifications; hence its properties do not vary nearly as much as those of barbed wire.

Wire rope

92. The tensile strengths and unrestrained moduli of military wire ropes depend primarily on their type of centers (hemp or steel); however, use (due to settling the strands around the steel or into the hemp centers) is known to increase the unrestrained modulus;[2] hence the moduli may vary within a wide range.

93. The tensile strengths of the specimens tested during this investigation ranged from 90,950 psi (6394 kg/cm^2) to 99,000 psi (6960 kg/cm^2) with their tensile moduli varying from 6,170,000 psi (433,794 kg/cm^2) to 9,431,000 (663,065 kg/cm^2), and in each case the higher values were found for the specimen containing a steel center.

[illegible] to the likelihood of damage to equipment, it was not feasible to obtain complete stress-strain curves; however, from the behavior of the wire ropes under loads, it is expected that the stress-strain curves of the tested wire ropes are fairly linear up to approximately 90 percent of their tensile strength.

95. Due to the diameter contraction of wire rope under loads, bond could be a problem, particularly for ropes with large diameters and hemp centers. This problem can be aggravated by incomplete removal of grease or oil. Thus, the use of ropes with diameters in excess of 5/8 in. (1.59 cm) to 3/4 in. (1.90 cm) is not recommended for expedient field concrete reinforcement.

Landing mat

96. An average yield strength (0.20-percent-offset method) of 48,000 psi (3374.7 kg/cm^2), an average tensile strength of 52,965 psi (3723.8 kg/cm^2), and an average elastic modulus of 28,750,000 psi

(2,021,326 kg/cm^2) were found on the samples of M8 pierced steel landing mat tested.

97. Since holes and protrusions in the landing mats provide for mechanical interlocking with the concrete, bond was not expected to be a problem, and no bond problems were experienced in the two beam tests conducted with this type of reinforcement.

Landing mat tie bars

98. The test results indicated an average yield strength of 45,390 psi (3191.2 kg/cm^2), an average tensile strength of 69,310 psi (4873.0 kg/cm^2), and an average tensile modulus of 28,983,300 psi (2,037,729 kg/cm^2) for six AM2 landing mat tie bar specimens tested. (Four specimens were tested maintaining their original paint, and two were tested with their paint removed by burning with an acetylene torch and brushing with a wire brush.) And, as all individual test results compared within acceptable limits (maximum standard deviation from the mean of approximately 5 percent), it was concluded that removing the bar's paint, as described, did not affect any of the above-mentioned properties.

99. The average bond strength of the bars retaining their original paint was 165 psi (11.6 kg/cm^2), but after removal of the paint the bond strength rose to 388 psi (27.3 kg/cm^2); therefore, only bars with their paint removed are recommended for concrete reinforcement.

Fabrication, Flexural Strength, Shear Strength, Cracking, and Deflections

Barbed or concertina wire

100. For rectangular beam sections, the range of practical reinforcement ratios (area of reinforcement to net beam cross-sectional area) appears to be approximately from 0.5 to 1.0 percent. Higher percentages result in overcrowded sections, while lower percentages appear to give unsatisfactory strengths. The strands must be placed rather closely to accommodate either ratio (as closely as 1/2 in. (1.27 cm) in the higher ratios); therefore, a maximum size aggregate of 3/8 in. (0.95 cm) is recommended for all concrete members in which either barbed or concertina wire is used as reinforcement.

101. Although assembling the individual wires resulted in a slightly smaller effective depth, and thus in a slightly lower flexural strength of the beams, this method is recommended for its practicality and substantial saving in labor.

102. All tests indicate that stirrups fabricated from either U. S. high tensile steel barbed or concertina wire make an efficient expedient shear reinforcement.

103. Due to the lower tensile modulus of barbed or concertina wire, structural elements reinforced with either material will generally have larger cracks and deflections than comparable members reinforced with conventional rebars. However, all tests indicated that if members are reinforced with suitable amounts of either wire (paragraph 100), deflections or cracks should not be a problem in temporary military construction.

Wire rope

104. In view of the potential bond problems with larger wire rope, the use of ropes with diameters in excess of 3/4 in. (1.90 cm) for expedient reinforcement is not recommended at this time. Careful removal of oil and grease prior to use is essential.

105. In order to keep beam deflections and cracking within tolerable limits, the low tensile modulus of wire rope should be compensated for with a high reinforcement ratio, equal to or greater than the maximum $0.75p_b$ ratio permitted by the ACI Code.[5] This technique of overreinforcing will, however, result in beams with very poor ductility.

106. To ensure utilization of a large portion of the tensile strength of wire rope, beams should be rather deep and preferably have a T section.

107. The above paragraphs indicate that wire rope is not an ideal substitute reinforcement and should perhaps be looked upon as an expedient reinforcing material of last resort.

108. Adequate shear protection can be provided for beams reinforced with wire rope by stirrups fabricated from either barbed or concertina wire.

Landing mat

109. Amounts of reinforcement equal to or somewhat less than that

specified by section 1601 of the ACI Code[5] are recommended for beams reinforced with sections of M8 pierced steel landing mat. When such amounts of reinforcement are properly placed into beams of adequate depths, all or practically all of the reinforcement's yield strength can generally be used effectively (which is a requirement for the recommended design and analysis procedures).

110. A concrete with a maximum size aggregate of 3/8 in. (0.95 cm) is recommended to ensure a satisfactory cover around the relatively small radii of curvature occurring at both the top and bottom of each section of reinforcement.

111. Due to the mechanical interlock provided by the holes and protrusions of the landing mats and due to their high tensile modulus (when compared to other materials such as barbed wire, concertina wire, and wire rope), neither cracks nor deflections should present any particular problems in temporary military construction.

112. Although sections of landing mat placed in a rectangular beam section in an upright position appear to be an effective partial shear reinforcement and have in fact prevented shear failures that would have normally occurred in both tested beams, it is at this time recommended to use additional expedient shear reinforcement as a safeguard against premature and brittle failures.

Landing mat tie bars

113. The flexural strength of comparable beams can be improved as much as 50 percent by simply removing the bar's original paint (during this study, the paint was removed with the aid of an acetylene torch and wire brush); therefore, bars retaining any type of paint are not recommended as reinforcement in any circumstances.

114. Beams which are reinforced with discarded AM2 landing mat tie bars should be provided with properly prepared (all bars should have their paint removed for greater bonding strengths) reinforcement in amounts equal to or very near that specified by section 1601 of the ACI Code.[5]

115. Protection against shear failures can be provided by stirrups fabricated from either high tensile steel barbed or concertina wire. Generally, either type requires a rather close spacing; therefore, a concrete

utilizing a maximum size aggregate greater than 3/8 in. (0.95 cm) is not recommended for this type of beam fabrication.

116. Again, as with most other expedient reinforcing materials mentioned in this report, deflections or excessive cracking should not present a particular problem when the beams are used in temporary military construction.

Analysis and Design

Barbed wire, concertina wire, wire rope, or landing mats

117. The suggested procedure for designing concrete beams reinforced with either of the mentioned reinforcing materials is as follows.

118. Flexural design. Select a beam cross section and reinforce it with the previously suggested reinforcement ratio (for either of the mentioned materials) and then analyze the section by an ultimate strength analysis (illustrated in Appendix A) recommended by N. C. Sinha and P. M. Ferguson[7] for concrete members reinforced with a high-strength steel having an indefinite yield point. This procedure is based on the following assumptions:

- a. Plane sections remain plane.
- b. There is no slip between concrete and reinforcement.
- c. Concrete will not take any tensile force.
- d. The concrete ultimate compressive strain is 0.0035.
- e. An equivalent rectangular concrete stress block (Whitney) can be assumed for the concrete in compression.
- f. The information concerning the reinforcement stress-strain curve is available.
- g. Minimum load factors are 1.5 for dead loads and 1.8 for the live loads (ultimate strength design loads given by the ACI Code[5]).

If the selected section does not have the desired strength, the section must be modified and the procedure repeated.

119. Design for shear. The above procedure is recommended for the flexural design; however, sections 1703 (using f_y = the 0.20-percent-offset yield strength) and 1706 of the ACI Code[5] appear to be adequate for

determining the required shear reinforcement for all beams.

120. Combined design. The above combined design procedure is recommended because it has predicted the load-carrying capabilities of all tested beams with adequate accuracy, whereas almost all of the individual test results indicated that a flexural design based on ultimate strength design procedures using a yield strength determined by the 0.20-percent-offset method failed to yield satisfactory results, especially for beams reinforced with barbed wire, concertina wire, and wire rope.

121. Several points, however, should be reemphasized.

a. If the in-air stress-strain curve of the chosen expedient reinforcing material shows an elastic modulus substantially below 30,000,000 psi (21,092,100 kg/cm^2), the restraining effects of embedment in concrete should be kept in mind. Depending on beam geometry and reinforcement ratio, a given difference between in-air and embedded behavior of the reinforcement may cause a variable error in the predicted moment capacity.

b. The conclusions concerning barbed wire are based on results of tests with U. S. high tensile steel barbed wire. Since there are reportedly more than 500 brands of barbed wire, some of which have poor strength properties, the conclusion can certainly not be expected to apply to all kinds of barbed wire.

c. Similar caution is necessary in the case of wire rope, although contrary to the barbed wire, the tested wire ropes appear to be on the lower end of reported range of properties,[2] at least as far as the elastic moduli are concerned. Of all the tested expedient materials, wire rope appears most apt to give rise to deflection, cracking, and bond problems. For this reason only small diameter ropes (diameters not in excess of 3/4 in.) should be used (if wire rope is used at all) and only after careful cleaning. To keep deflections and cracks in reasonable limits, the reinforcing ratios should exceed those allowed by the code, although this will, of course, result in poor ductility.

d. In beams reinforced with sections of pierced steel landing mat (which have a definite yield point), ACI Code USD may be used, provided the reinforcement ratio and the h/d ratios are small enough to ensure yielding of all reinforcement steel prior to compression failure.

Landing mat tie bars

122. Either of the following flexural designs combined with the recommended shear design procedure appears to be satisfactory for the design of

beams reinforced with AM2 landing mat tie bars (both designs are based on the assumption that all paint is removed from the tie bar).

123. Flexural design. Either the working stress or ultimate strength design recommended by the ACI Code 318-63[5] for the flexural design of concrete members reinforced with plain bars can be used as a guide in designing beams reinforced with discarded AM2 landing mat tie bars; however, the beam tests indicated that the code's allowable bond strength of 160 psi or 11.25 kg/cm^2 (working stress design) and 250 psi or 17.58 kg/cm^2 (ultimate strength design) should be reduced by about 20 percent.

124. Design for shear. The most practical method of providing expedient shear reinforcement appears to be the use of stirrups fabricated from either high tensile barbed or concertina wire; therefore, the previously recommended procedure for the design of either barbed or concertina wire stirrups is again recommended for beams reinforced with discarded AM2 landing mat tie bars.

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Table 1

Yield Strength, Tensile Strength, and Modulus of Elasticity of Barbed and Concertina Wire

Specimen No.		Yield Strength (0.20-percent-Offset Method) psi	Yield Strength (0.20-percent-Offset Method) kg/cm^2	Tensile Strength psi	Tensile Strength kg/cm^2	Tensile Modulus psi	Tensile Modulus kg/cm^2
			Complete Strand of Barbed Wire				
BW1		71,000	4,991.8	99,000	6,960.4	21,053,000	1,480,173
BW2		77,000	5,413.6	100,100	7,037.7	16,000,000	1,124,912
BW3		77,000	5,413.6	102,500	7,206.5	21,333,000	1,499,859
	Avg	75,000	5,273.0	100,533	7,068.2	19,462,000	1,368,315
			Concertina Wire				
CW1		150,000	10,546.1	204,350	14,367.2	26,446,300	1,859,360
CW2		141,000	9,913.3	203,480	14,306.1	27,586,200	1,939,503
CW3		148,000	10,405.4	202,610	14,244.9	23,728,800	1,668,301
	Avg	146,333	10,288.2	203,480	14,306.1	25,920,433	1,822,388

Note: Diameter of a single wire of a barbed wire strand was 0.089 in. (0.23 cm); each complete strand consisted of two individual wrapped wires. The diameter of the concertina wire was 0.121 in. (0.31 cm).

Table 2

Tensile Strength and Modulus of Elasticity of Typical United States Military Wire Ropes

Specimen No.	Diameter		Wire Rope Type	Tensile Strength		Tensile Modulus		Remarks
	in.	cm		psi	kg/cm^2	psi	kg/cm^2	
1	1.250	3.18	6 wires; steel center	99,000	6960	9,431,000	663,065	Tank retriever cable
2	0.625	1.59	6 wires; hemp center	93,810	6595	6,170,000	433,794	Jeep wrecker cable
3	0.750	1.90	6 wires; hemp center	90,950	6394	7,750,000	544,879	Railway tie cable

Table 3

Yield Strength, Tensile Strength, and Modulus of Elasticity of M8 Pierced Steel Landing Mats

Specimen No.	Thickness of Specimens		Yield Strength		Tensile Strength		Tensile Modulus		Remarks
	in.	cm	psi	kg/cm^2	psi	kg/cm^2	psi	kg/cm^2	
1	0.140	0.36	48,000	3374.7	52,320	3678.5	28,000,000	1,968,596	Specimen 1 was obtained from panel 3 of bundle 1
2	0.139	0.35	48,000	3374.7	53,610	3769.2	29,500,000	2,074,057	Specimen 2 was obtained from panel 1 of bundle 2
		Avg	48,000	3374.7	52,965	3723.8	28,750,000	2,021,326	

Table 4

Yield Strength, Tensile Strength, and Modulus of Elasticity of AM2 Landing Mat Tie Bars

Specimen No.		Yield Strength		Tensile Strength		Tensile Modulus		Remarks
		psi	kg/cm^2	psi	kg/cm^2	psi	kg/cm^2	
1		44,900	3156.8	68,660	4827.3	29,600,000	2,081,087	Paint not removed
2		45,500	3199.0	69,000	4851.2	30,500,000	2,144,364	Paint not removed
3		41,500	2917.7	66,880	4702.1	28,000,000	1,968,593	Paint not removed
4		47,000	3304.4	72,870	5123.3	29,500,000	2,074,057	Paint not removed
5		46,940	3300.2	69,040	4854.0	28,100,000	1,975,627	Paint removed*
6		46,500	3269.3	69,430	4881.4	28,200,000	1,982,657	Paint removed*
	Avg	45,390	3191.2	69,310	4873.0	28,983,300	2,037,729	

Note: All specimens were 1.00 in. (2.54 cm) in diameter.

* Paint was removed by burning with an acetylene torch and brushing with a wire brush.

Table 5

Ultimate Bond Strength of AM2 Landing Mat Tie Bars

Paint Not Removed

Specimen No.	Batch No.	Batch Cylinder Strength* psi	Batch Cylinder Strength* kg/cm^2	Average Bond Strength** psi	Average Bond Strength** kg/cm^2
1	1	3190	224.3	142	10.0
2				125	8.8
3				139	9.8
4				158	11.1
5				152	10.7
6				109	7.7
7				201	14.1
8				252	17.7
9				134	9.4
10				191	13.4
11				188	13.2
12				184	12.9
			Avg	165	11.6

Paint Removed†

Specimen No.	Batch No.	Batch Cylinder Strength* psi	Batch Cylinder Strength* kg/cm^2	Average Bond Strength** psi	Average Bond Strength** kg/cm^2
13	2	3260	229.2	402	28.3
14				446	31.4
15				317	22.3
			Avg	388	27.3

Note: Range of specimens retaining their original paint is 109 psi ($7.7\ kg/cm^2$) to 252 psi ($17.7\ kg/cm^2$). Range of specimens with paint removed is 317 psi ($22.3\ kg/cm^2$) to 446 psi ($31.4\ kg/cm^2$).

* Average of three 28-day tests on 3- by 6-in. (7.62- by 15.24-cm) cylinders.

** Ultimate 28-day pullout bond strength between 1.00-in.- (2.54-cm-) diameter tie bars and 6-in. (15.24-cm) concrete cubes.

† Paint removed by burning with an acetylene torch and brushing with a wire brush.

Table 6

Concrete Mixture Data for a One-Bag Batch

Material	Volume		Weight	
	cu ft	m^3	lb	kg
Type II cement	0.479	0.0136	94.0	42.6
Fine limestone aggregate	2.154	0.0610	358.3	162.5
Coarse limestone aggregate	2.069	0.0586	349.4	158.5
Water	1.297	0.0367	80.8	36.6

Admixtures: None
Water-cement ratio by weight: 0.86
Slump: 2 $\pm$ 1/2 in. (5.08 $\pm$ 1.27 cm)
Cement content: 4.5 bags/cu yd (250 kg/m^3)

Note: See tables 7, 8, 9, 10, and 12 for the strength data of each individual batch.

Table 7

Results of Flexure Test of Concrete Beams Reinforced with Either Barbed or Concertina Wire

Specimen No.	Concrete Age When Tested (day)	28-Day Concrete Cylinder Strength (psi)	28-Day Concrete Cylinder Strength (kg/cm²)	Strands of Reinforcement per Beam	Percent Reinforcement (Net Section)	Midspan Deflection at Maximum Load (in.)	Midspan Deflection at Maximum Load (cm)	Maximum Moment (Tested) (in.-lb)	Maximum Moment (Tested) (m-kg)	Theoretical Ultimate Moment (Disregarding Shear Failure) (in.-lb)	Theoretical Ultimate Moment (Disregarding Shear Failure) (m-kg)	Moment Corresponding to Ultimate Shear Strength According to ACI Code Equation 17-2 with ϕ = 1 (in.-lb)	Moment Corresponding to Ultimate Shear Strength According to ACI Code Equation 17-2 with ϕ = 1 (m-kg)	Calculated Mean Stress in Reinforcement at Failure Using an Ultimate Strength Analysis (psi)	Calculated Mean Stress in Reinforcement at Failure Using an Ultimate Strength Analysis (kg/cm²)	Type of Failure	Remark
								Group 1									
BW1	28	3180	223.6	12 (barbed wire)	0.465	1.00	2.54	105,120	1211.1	102,235	1177.8	91,380	1052.8	93,300	6,559.6	Shear	Not reinforced for shear
BW2	28	3180	223.6	24 (barbed wire)	0.960	0.75	1.90	132,000	1520.8	158,290	1823.7	97,590	1124.3	[illegible]	[illegible]	Shear	Not reinforced for shear
								Group 2									
BW3	28	4196	295.0	12 (barbed wire)	0.465	1.50	3.81	120,000	1382.5	108,280	1247.5	--	--	97,100	6,826.8	Concrete crushed in compression	Reinforced for shear with barbed wire stirrups
BW4	[illegible]	4196	295.0	24 (barbed wire)	0.960	1.25	3.18	186,000	2142.9	174,980	2015.9	--	--	[illegible]	[illegible]	Concrete crushed in compression	Reinforced for shear with barbed wire stirrups
								Group 3									
CW1	[illegible]	3460	243.3	12 (concertina wire)	0.431	1.25	3.18	157,200	1811.1	155,460	1791.1	--	--	159,500	11,214.[illegible]	Concrete crushed in compression	Reinforced for shear with concertina wire stirrups
								Group 4									
CW2	28	[illegible]	[illegible]	24 (concertina wire)	0.920	1.00	2.54	236,400	2723.6	198,600	2288.1	--	--	[illegible]	[illegible]	Concrete crushed in compression	Reinforced for shear with concertina wire stirrups
BW5	28	[illegible]	[illegible]	24 (barbed wire)	0.960	1.70	4.32	169,200	1949.4	159,480	1837.4	--	--	[illegible]	[illegible]	Concrete crushed in compression	Reinforced for shear with barbed wire stirrups

Note: Results for third-point loading of 4- by 9- by 78-in. (10.16- by 22.86- by 198.12-cm) beams.

Table 3

Results of Flexure Test of Concrete Beams Reinforced with U. S. Military Wire Rope, Group 5

Specimen No.	Concrete Age When Tested	28-Day Concrete Cylinder Strength		No. of Wire Ropes per Beam	Diameter of Wire Ropes		Percent Reinforcement (Net Section)	Midspan Deflection at Maximum Load		Maximum Moment (Tested)		Theoretical Ultimate Moment[7]		Calculated Mean Stress in Reinforcement at Failure Using an Ultimate Strength Analysis[7]		Type of Failure	Remarks
	days	psi	kg/cm²		in.	cm		in.	cm	in.-lb	m-kg	in.-lb	m-kg	psi	kg/cm²		
Beam WR1	27	3720	261.5	2	0.750	1.90	2.760	0.860	2.18	261,600	3013.9	231,210	2663.8	39,500	2777.1	Flexural compressive	Beam reinforced for shear with concertina wire stirrups
Beam WR2	28	3720	261.5	2	0.625	1.59	1.916	1.230	3.12	190,800	2198.2	173,000	1993.1	39,800	2798.2	Flexural compressive	Beam reinforced for shear with concertina wire stirrups

Note: Results for third-point loading of 4- by 9- by 78-in. (10.16- by 22.86- by 198.12-cm) beams.

Table 9

Results of Flexure Test of Concrete Beams Reinforced with Sections of M8 Pierced Steel Landing Mat

Specimen No.	Concrete Age When Tested	28-Day Concrete Cylinder Strength		Sections of Mat per Beam	Percent Reinforcement (Net Section)	Midspan Deflection at Maximum Load		Maximum Moment (Tested)		Theoretical Ultimate Moment[7]		Calculated Mean Stress in Reinforcement at Failure Using an Ultimate Strength Analysis[7]		Type of Failure
	days	psi	kg/cm^2			in.	cm	in.-lb	m-kg	in.-lb	m-kg	psi	kg/cm^2	
Group 6 LM1	28	3720	261.5	1	2.936	0.700	1.78	150,000	1728.2	169,940	1957.9	46,500	3269.3	Flexural compressive
Group 7 LM2	28	3540	248.9	3	8.089	0.490	1.24	232,200	2675.5	215,920	2487.6	23,700	1666.3	Flexural compressive

Note: Results for third-point loading of 4- by 9- by 78-in. (10.16- by 22.86- by 198.12-cm) beams. Reinforcement was placed as shown in fig. 3c.

Table 11

Results of Flexure Tests of Concrete Beams Reinforced with AM2 Landing Mat Tie Bars

Specimen No.	Concrete Age at Test, day	28-Day Concrete Cylinder Strength, psi	28-Day Concrete Cylinder Strength, kg/cm²	Percent Reinforcement (Net Section)	Midspan Deflection at Maximum Load, in.	Midspan Deflection at Maximum Load, cm	Maximum Moment (Tested), in.-lb	Maximum Moment (Tested), m-kg	Ultimate Moment Using Ultimate Strength Design of ACI Code and Disregarding Shear and Bond, in.-lb	Ultimate Moment ..., m-kg	Moment Corresponding to Allowable Shear Using ACI Code Formula, in.-lb	Moment Corresponding to Allowable Shear ..., m-kg	Calculated Mean Bond Stress in Reinforcement*: Working Stress Analysis Using ACI Code Formula, psi	Working Stress Analysis, kg/cm²	Ultimate Strength Analysis Using ACI Code, psi	Ultimate Strength Analysis, kg/cm²	At Failure*: Elastic Analysis, psi	At Failure*: Elastic Analysis, kg/cm²	At Failure*: Modified Ultimate Strength Analysis, psi	At Failure*: Modified Ultimate Strength Analysis, kg/cm²	Type of Failure	Remarks
												Group 1										
AM2-1	[illegible]	29[illegible]	[illegible]	2.49	[illegible].33	[illegible]	22[illegible]	[illegible].4	54[illegible]	[illegible]29[illegible]	36[illegible]	4223.[illegible]	[illegible]	[illegible]	[illegible]3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible].2	Bond	Original paint not removed. Beam not reinforced for shear
AM2-2	[illegible]	[illegible]	[illegible]	2.44	[illegible]3	[illegible]	221,[illegible]	[illegible]1.4	54[illegible]	[illegible]29[illegible]	36[illegible],600	4223.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Bond	Same as beam AM2-1
												Group 2										
AM2-3	[illegible]	39[illegible]	[illegible]9.2	[illegible]	[illegible].43[illegible]	[illegible]	352.[illegible]	[illegible]55.4	[illegible]5,2[illegible]	[illegible]512.[illegible]	[illegible]	[illegible]338.[illegible]	[illegible]	[illegible].3	[illegible]	[illegible]	2[illegible]	[illegible]803.4	2[illegible]	[illegible]	[illegible]	Original paint removed with acetylene torch and wire brush. Beam not reinforced for shear
AM2-4	[illegible]	[illegible]	2[illegible]9.	[illegible]	1.[illegible]50	[illegible]	[illegible],2[illegible]	[illegible]8.1	[illegible],230	[illegible]	[illegible]	[illegible]	[illegible]	1[illegible].4	188	[illegible].2	39,[illegible]	2[illegible]	42,22[illegible]	2[illegible]8.4	Shear	Same as beam AM2-3
												Group 3										
AM2-5	[illegible]	[illegible]	[illegible]	2.49	[illegible].2[illegible]	3.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]982.[illegible]	--	--	20[illegible]	1[illegible].5	239	[illegible].8	3[illegible],2[illegible]	[illegible]40.3	3[illegible]	25[illegible].3	Flexural Bond	Original paint removed with acetylene torch and wire brush. Beam reinforced for shear with barbed wire stirrups

Note: Tests were conducted on 138- by 12- by 5-in. (350.52- by 30.48- by 12.70-cm) beams. Loading was at the third points except for beam AM2-4 which was loaded at midspan.

* See Appendix B for example using the modified ultimate strength and elastic analyses.

Table 11

Results of Tensile Test of Butt-Welded AM2 Landing Mat Tie Bar Splints

Specimen No.		Yield Strength (0.20-percent-Offset Method)		Tensile Strength		Percent Elongation	Remarks
		psi	kg/cm^2	psi	kg/cm^2		
1W		42,500	2988.0	56,430	3967.4	4.20	Specimen failed at center of weld
2W		42,750	3005.6	52,040	3658.8	3.40	Specimen failed at center of weld
3W		42,750	3005.6	54,140	3806.4	3.25	Specimen failed at center of weld
	Avg	42,670	3000.0	54,200	3810.6	3.62	
1C		43,000	3023.2	67,130	4719.7	9.00	Control specimen (not welded)

Note: Gage length was 8 in. (20.32 cm).

Table 12

Result of Flexure Test of Concrete Beams Reinforced with 1-1/4-in.- (3.18-cm-) Diameter U. S. Military Wire Rope

Specimen No.	Concrete Age When Tested	28-Day Concrete Cylinder Strength		No. of Wire Ropes per Beam	Percent Reinforcement (Net Section)	Midspan Deflection at Maximum Load		Maximum Moment (Tested)		Theoretical Ultimate Moment		Calculated Mean Stress in Reinforcement at Failure Using a Modified Ultimate Strength Analysis*		Calculated Mean Stress in Reinforcement at Failure Using an Elastic Analysis*	
	days	psi	kg/cm^2			in.	cm	in.-lb	m-kg	in.-lb	m-kg	psi	kg/cm^2	psi	kg/cm^2
Group 4															
WR3	28	3420	240.4	1	0.0130	1.50	3.81	290,000	3341.1	936,440	10,788.7	18,220	1281.0	18,960	1333.0

Note: Results for third-point loading of 180- by 15- by 7-in. (457.20- by 38.10- by 17.78-cm) beam.
* See Appendix B for example using the modified ultimate strength and elastic analyses.

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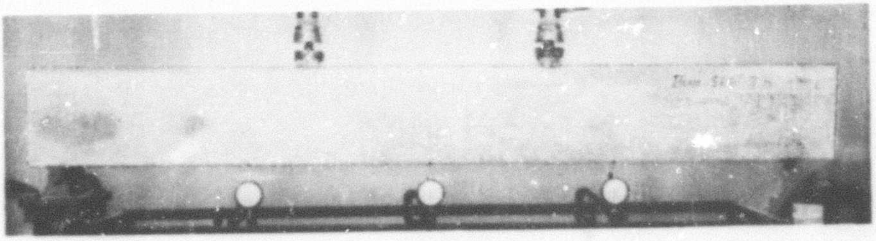
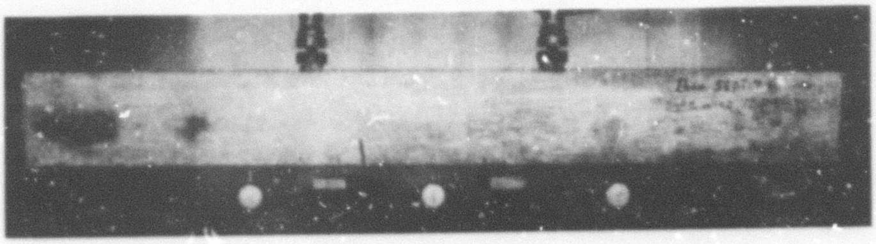
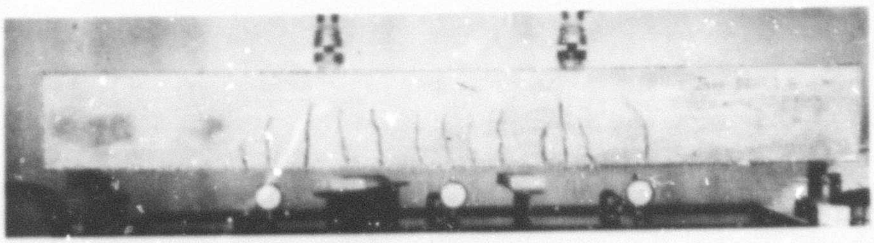


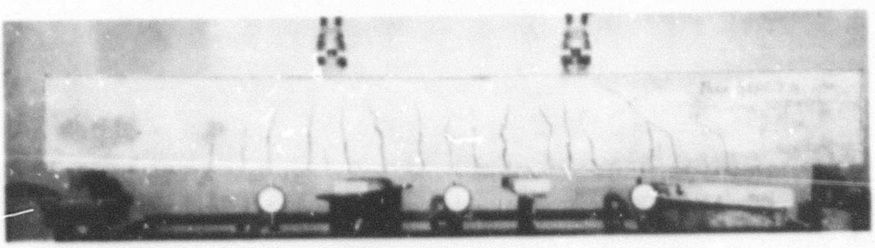
a. No load



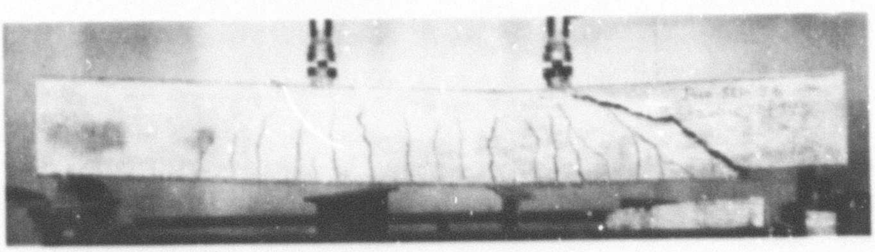
b. Total load = 3500 lb (1587.6 kg)



c. Total load = 4300 lb (1950.4 kg)

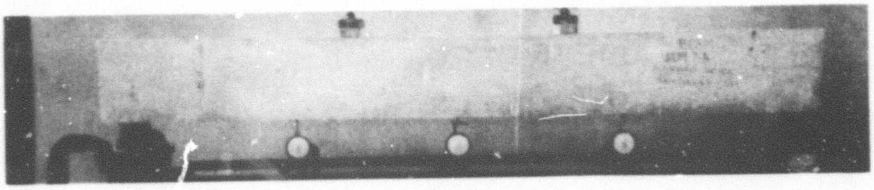


d. Total load = 7500 lb (3401.9 kg)

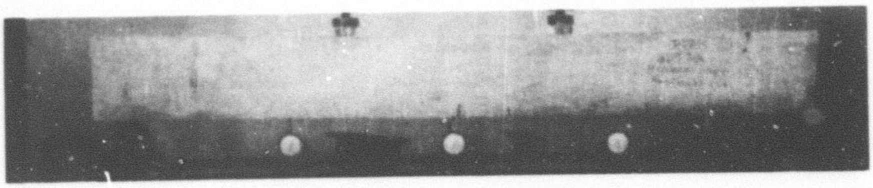


e. Total load = 8760 lb (3973.5 kg), failure

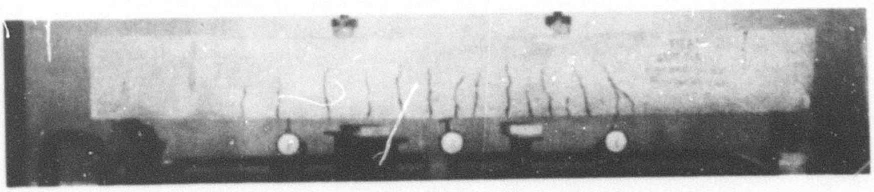
Photograph 1. Crack pattern, beam BW1



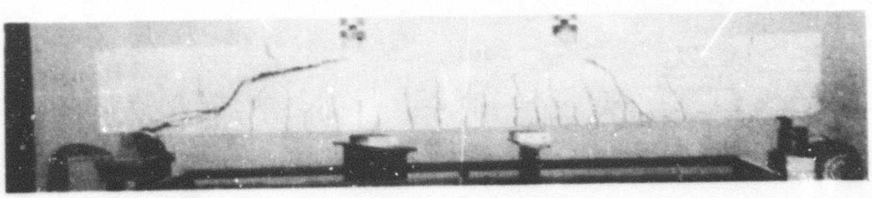
a. No load



b. Total load = 4000 lb (1814.4 kg)

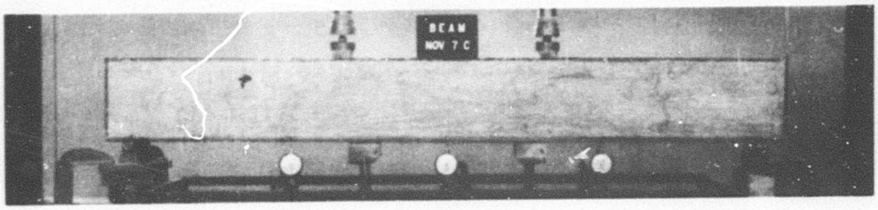


c. Total load = 9000 lb (4082.3 kg)

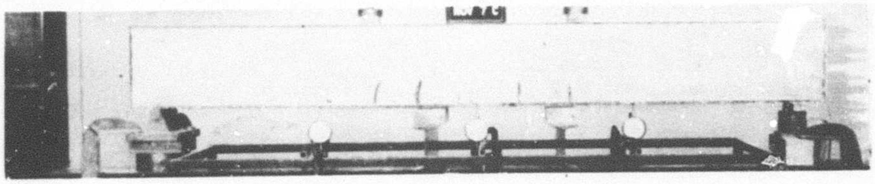


d. Total load = 11,000 lb (4989.5 kg), failure

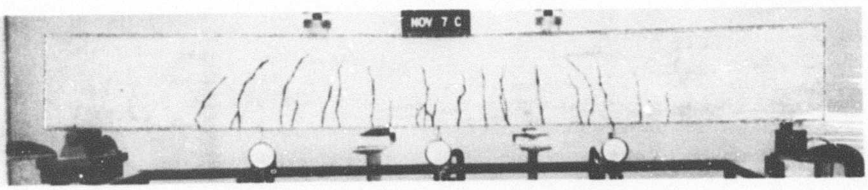
Photograph 2. Crack pattern, beam BW2



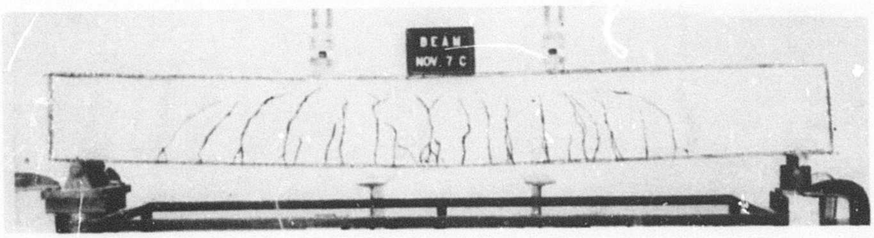
a. No load



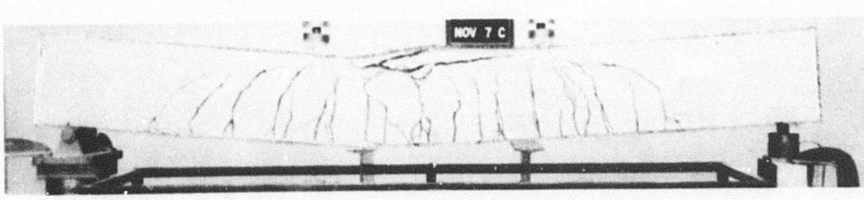
b. Total load = 3500 lb (1587.6 kg)



c. Total load = 8000 lb (3628.7 kg)

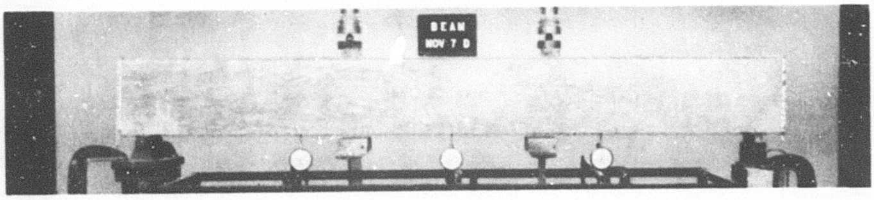


d. Total load = 9500 lb (4309.1 kg)

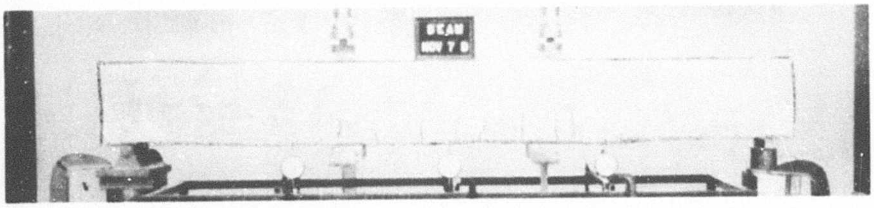


e. Total load = 10,000 lb (4535.9 kg), failure

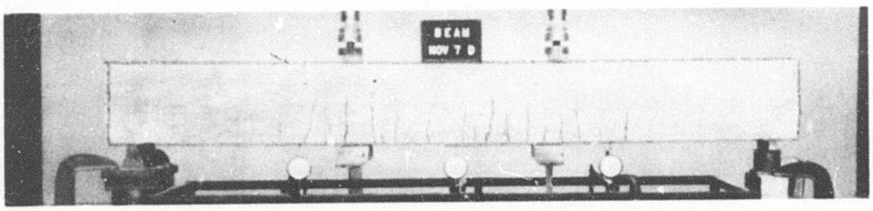
Photograph 3. Crack pattern, beam BW3



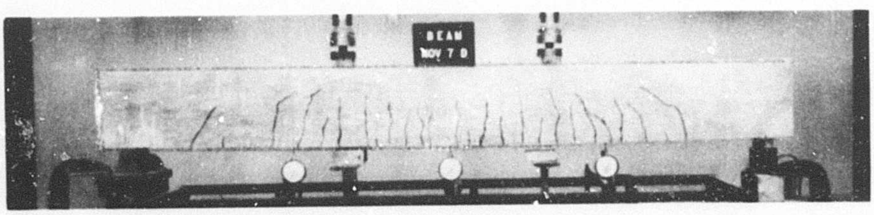
a. No load



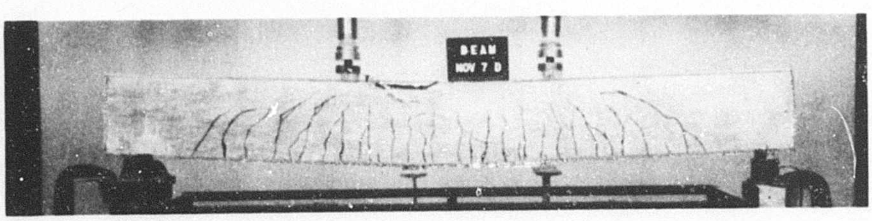
b. Total load = 4500 lb (2041.2 kg)



c. Total load = 8500 lb (3855.5 kg)

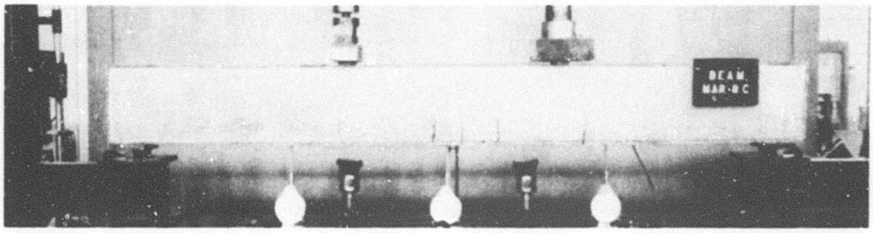


d. Total load = 12,000 lb (5443.1 kg)

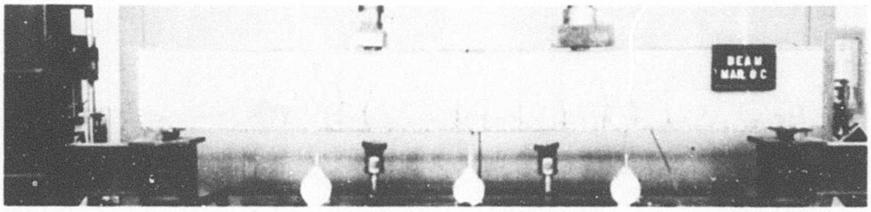


e. Total load = 15,500 lb (7030.7 kg), failure

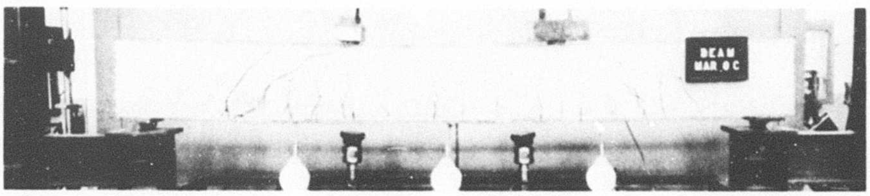
Photograph 4. Crack pattern, beam BW4



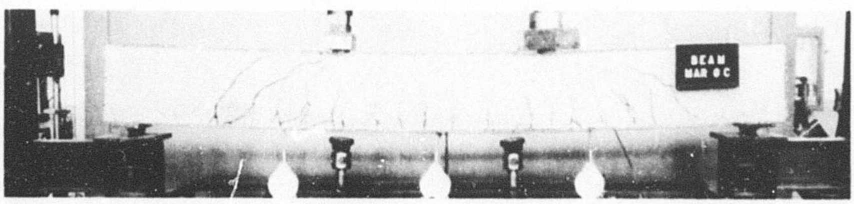
a. Total load = 3000 lb (1360.8 kg)



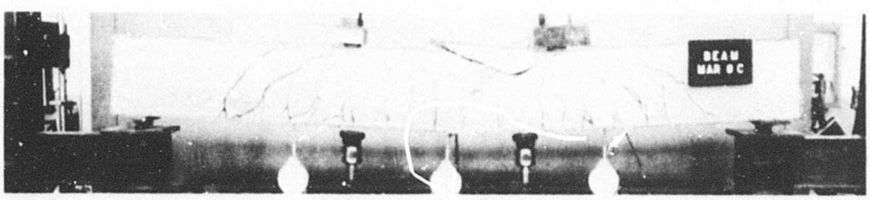
b. Total load = 5500 lb (2494.8 kg)



c. Total load = 10,000 lb (4535.9 kg)

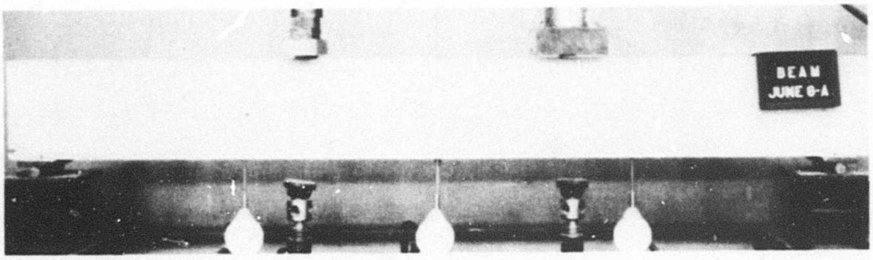


d. Total load = 12,000 lb (5443.1 kg)

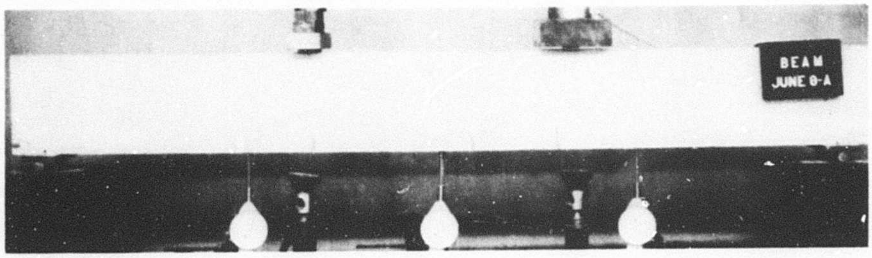


e. 0 load after failure

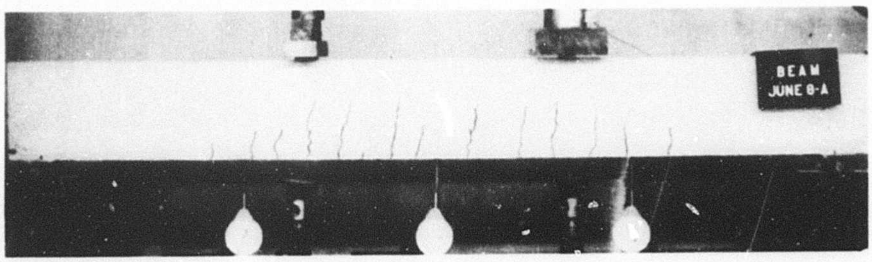
Photograph 5. Crack pattern, beam CW1



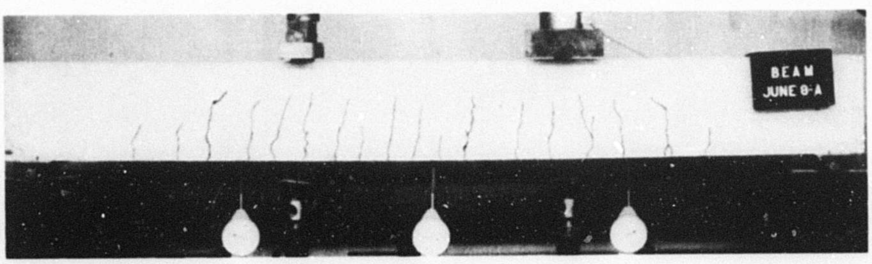
a. No load



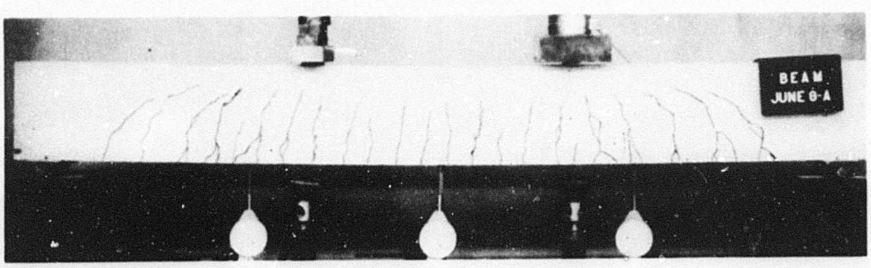
b. Total load = 3000 lb (1360.8 kg)



c. Total load = 5000 lb (2268.0 kg)

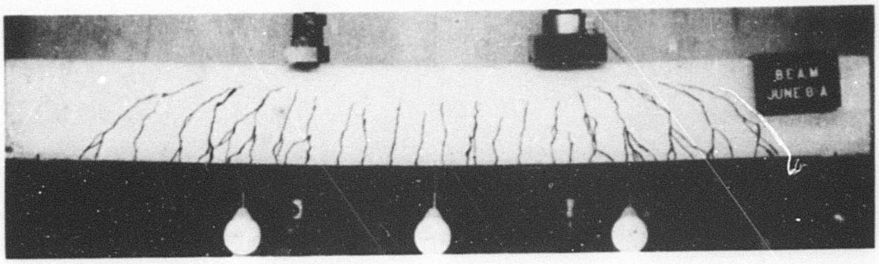


d. Total load = 8000 lb (3628.7 kg)

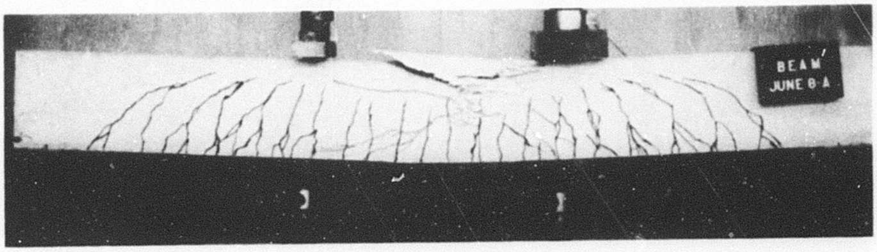


e. Total load = 13,000 lb (5896.7 kg)

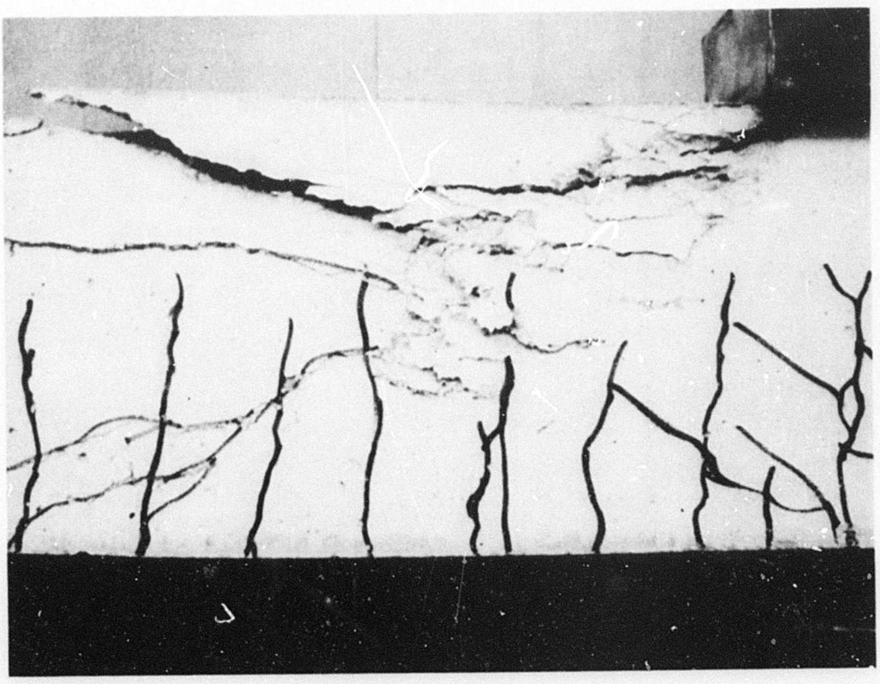
Photograph 6. Crack pattern, beam CW2 (1 of 2 sheets)



f. Total load = 18,000 lb (8164.7 kg)

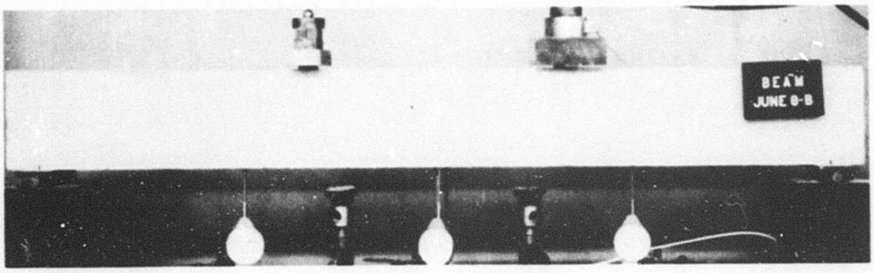


g. Total load = 19,700 lb (8935.8 kg), failure

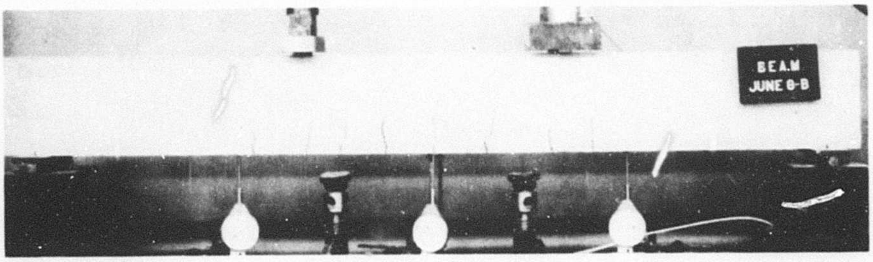


h. Closeup at failure

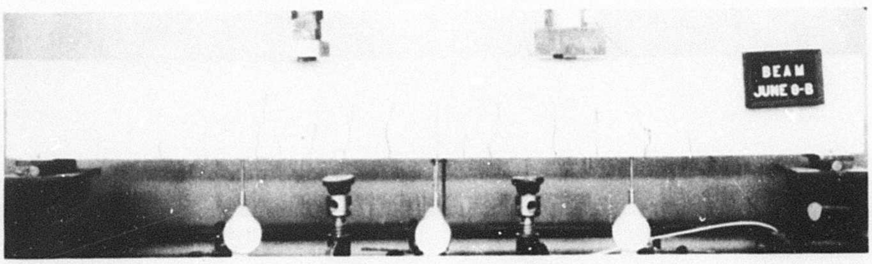
Photograph 6 (2 of 2 sheets)



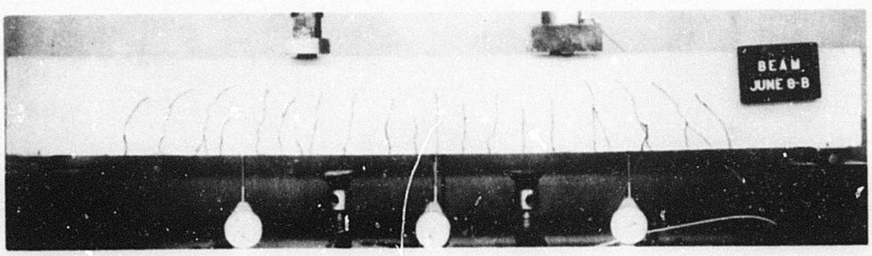
a. No load



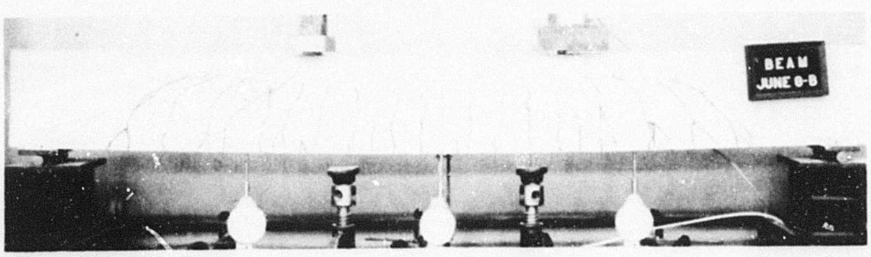
b. Total load = 4000 lb (1814.4 kg)



c. Total load = 6000 lb (2721.6 kg)

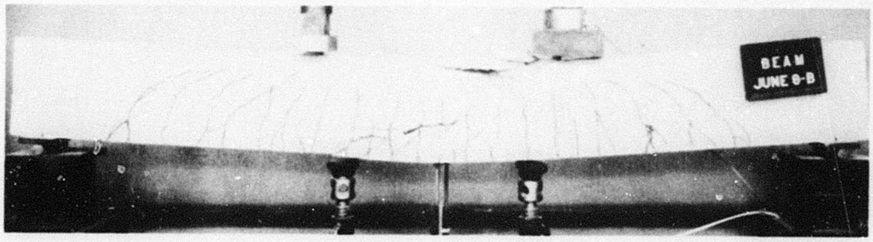


d. Total load = 11,000 lb (4989.5 kg)

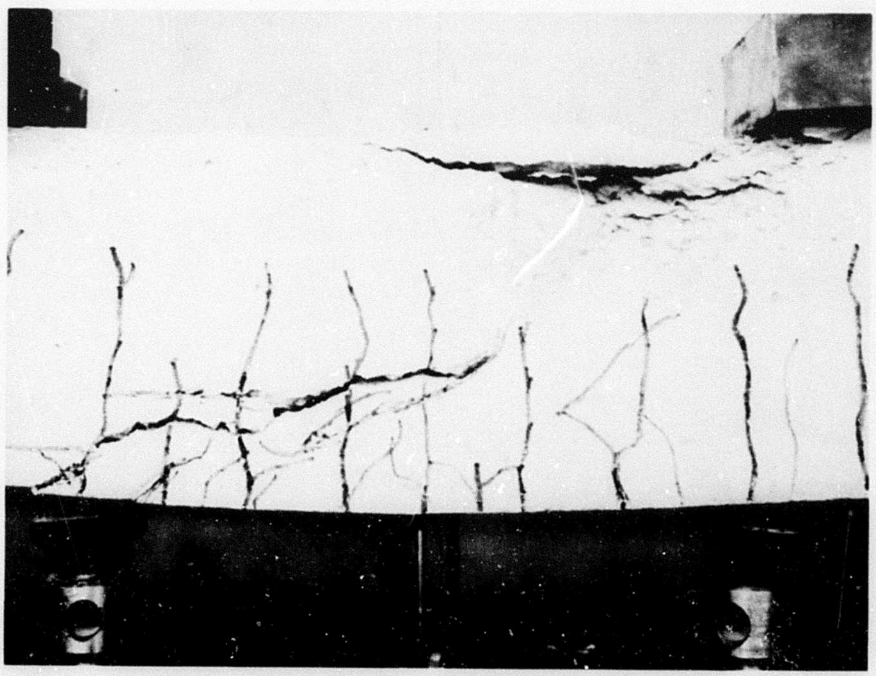


e. Total load = 13,000 lb (5896.7 kg)

Photograph 7. Crack pattern, beam BW5 (1 of 2 sheets)

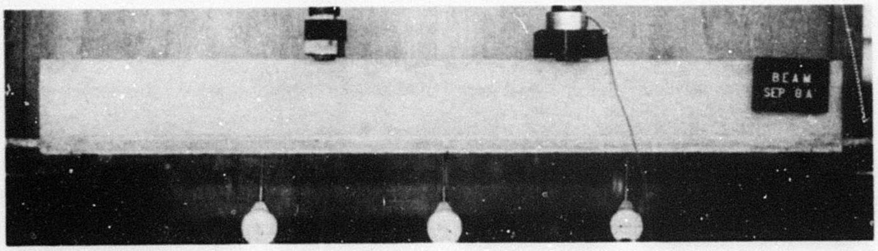


f. Total load = 14,100 lb (6395.7 kg), failure

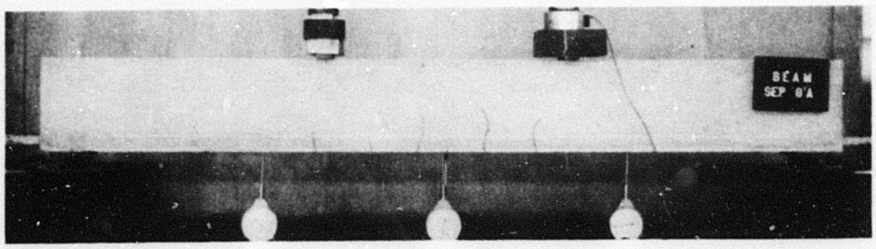


g. Closeup at failure

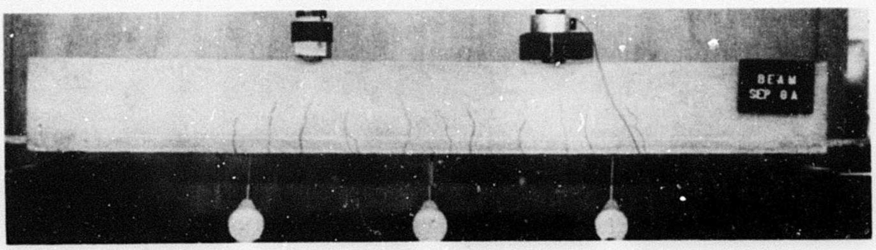
Photograph 7 (2 of 2 sheets)



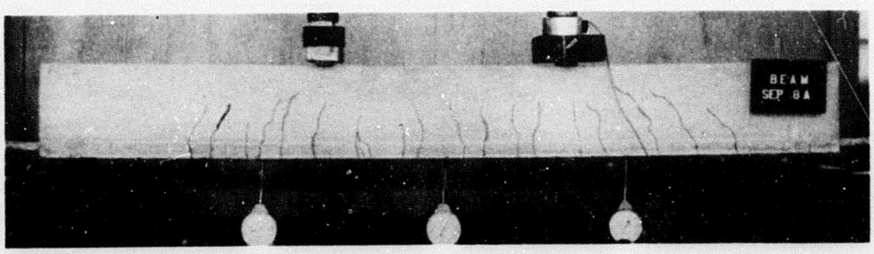
a. No load



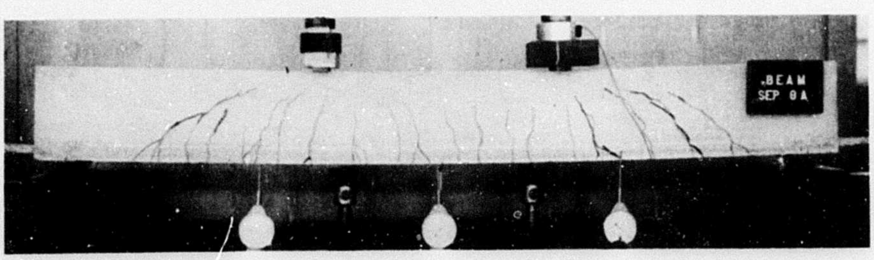
b. Total load = 5000 lb (2268.0 kg)



c. Total load = 8000 lb (3628.7 kg)

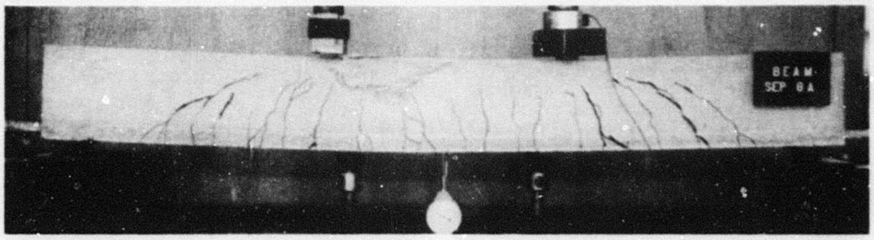


d. Total load = 12,000 lb (5443.1 kg)

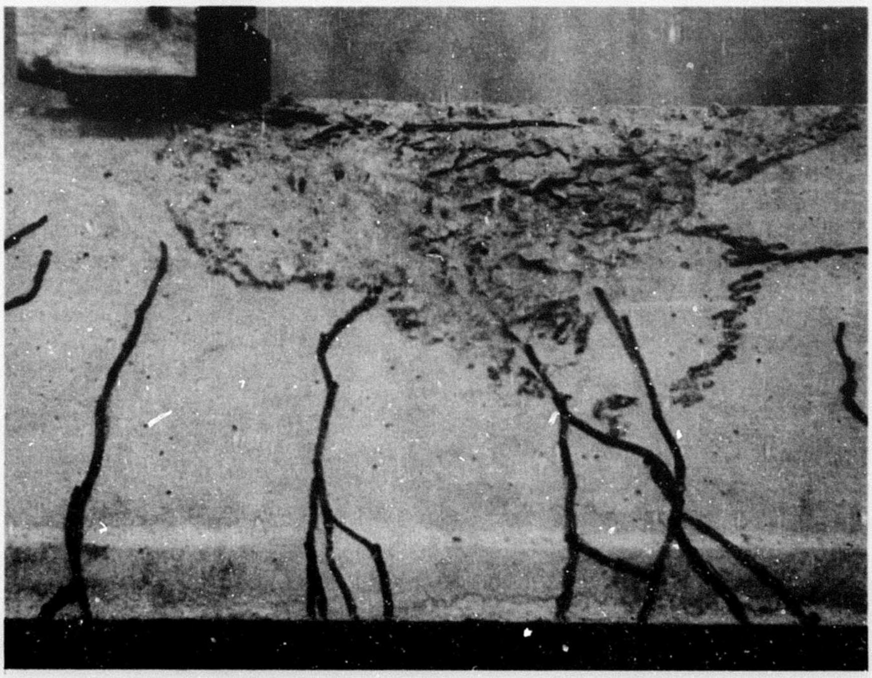


e. Total load = 20,000 lb (9071.9 kg)

Photograph 8. Crack pattern, beam WR1 (1 of 2 sheets)

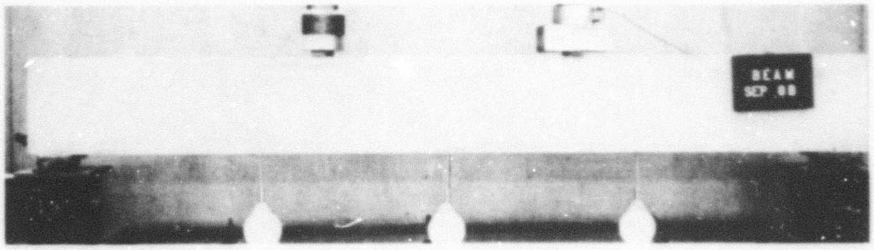


f. Total load = 21,800 lb (9888.3 kg), failure

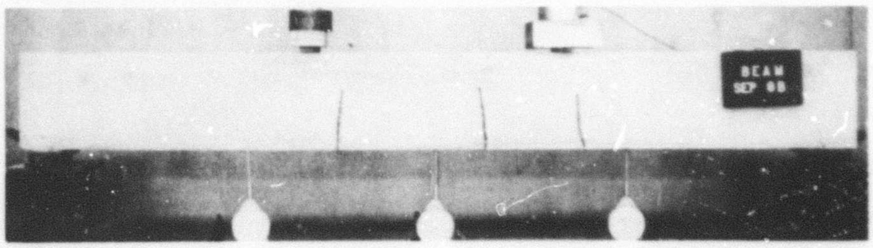


g. Closeup at failure

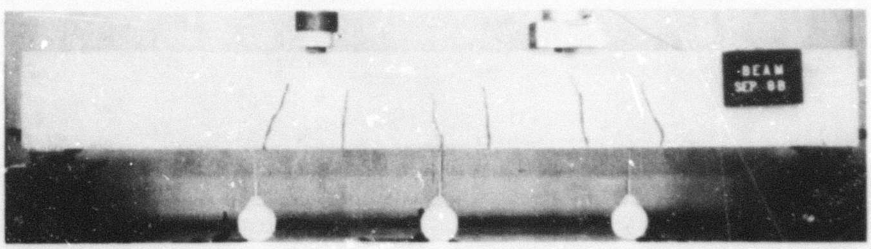
Photograph 8 (2 of 2 sheets)



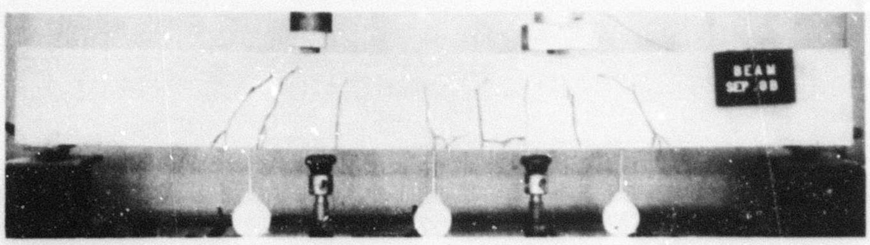
a. No load



b. Total load = 3000 lb (1360.8 kg)

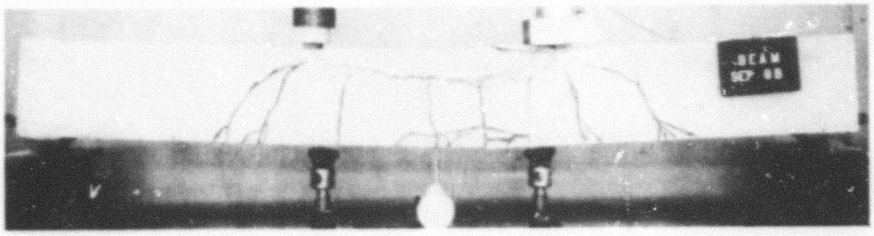


c. Total load = 5000 lb (2268.0 kg)

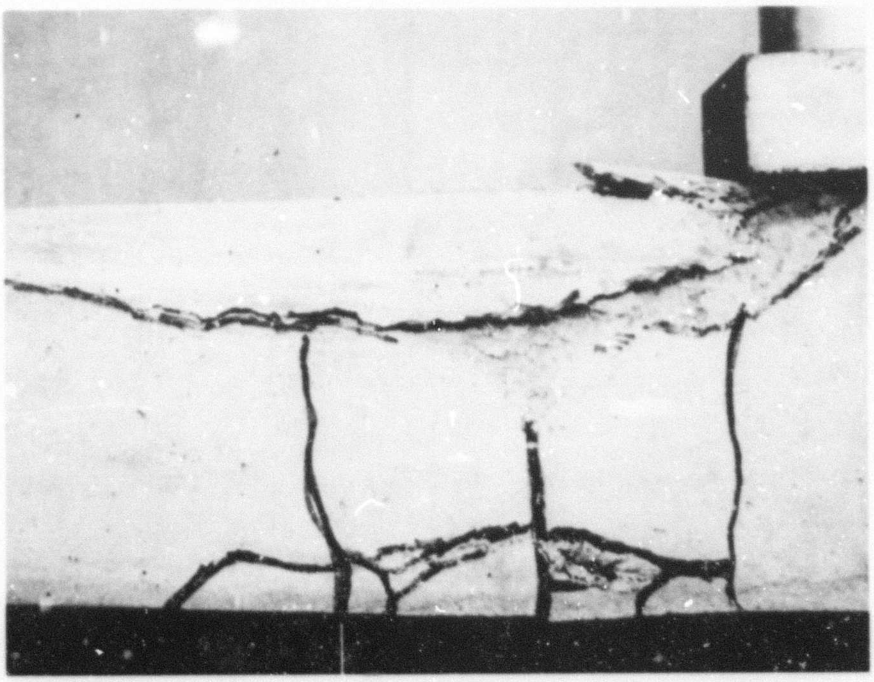


d. Total load = 10,000 lb (4535.9 kg)

Photograph 9. Crack pattern, beam WR2 (1 of 2 sheets)

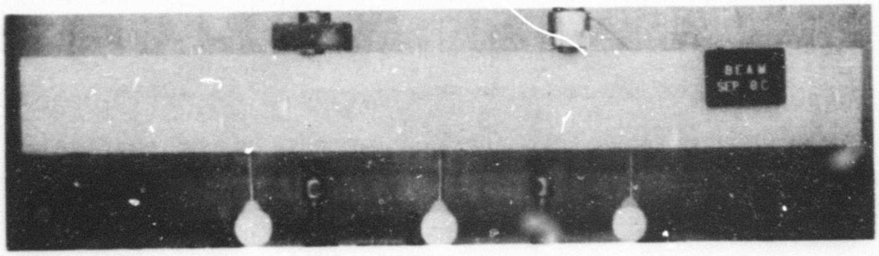


e. Total load = 15,900 lb (7212.1 kg), failure

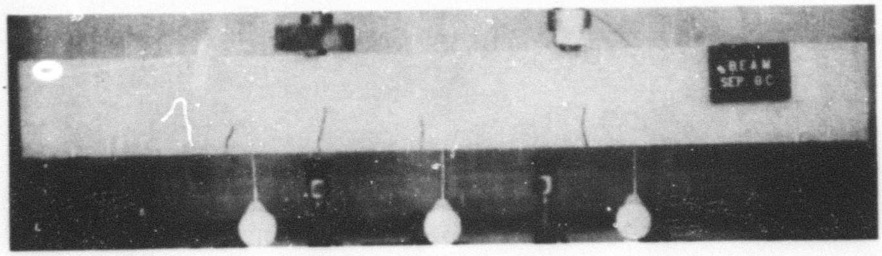


f. Closeup at failure

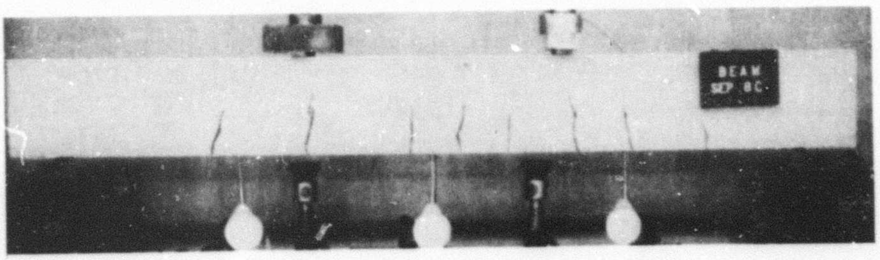
Photograph 9 (2 of 2 sheets)



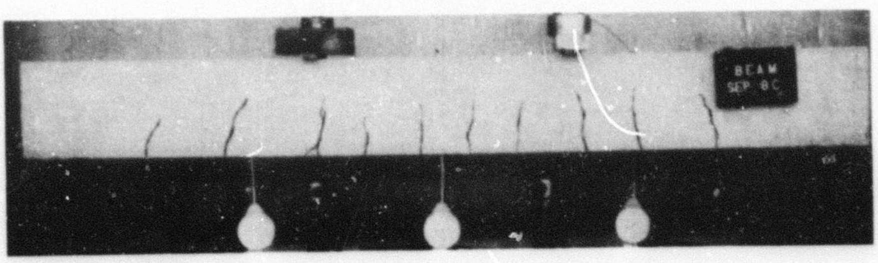
a. No load



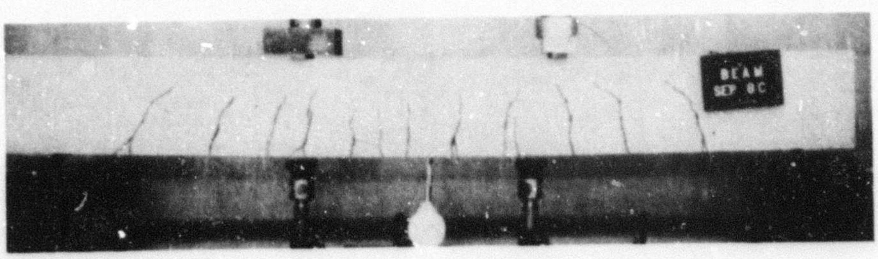
b. Total load = 3000 lb (1360.8 kg)



c. Total load = 5000 lb (2268.0 kg)

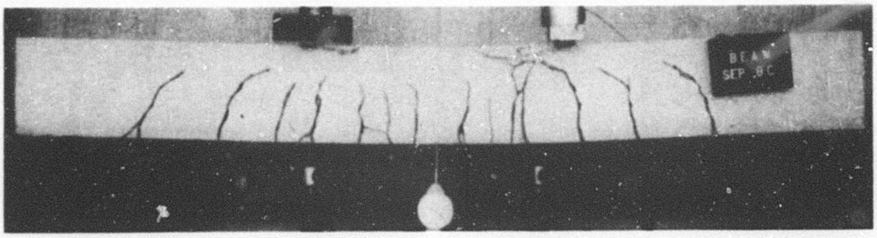


d. Total load = 8000 lb (3628.7 kg)

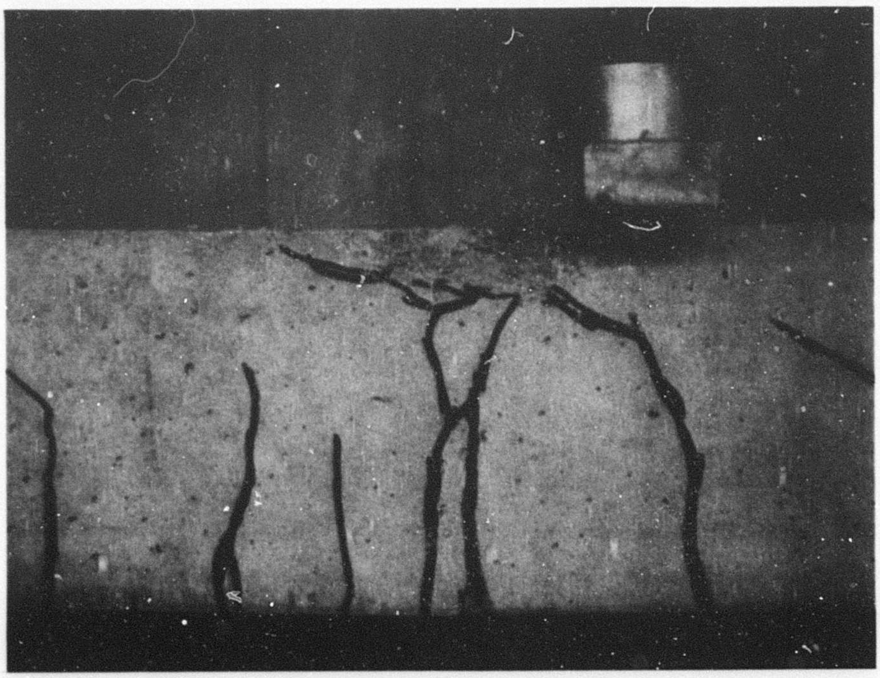


e. Total load = 12,000 lb (5443.1 kg)

Photograph 10. Crack pattern, beam LM1 (1 of 2 sheets)

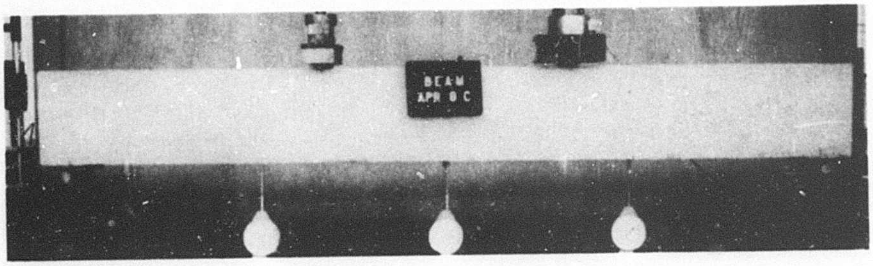


f. Total load = 12,500 lb (5669.9 kg), failure

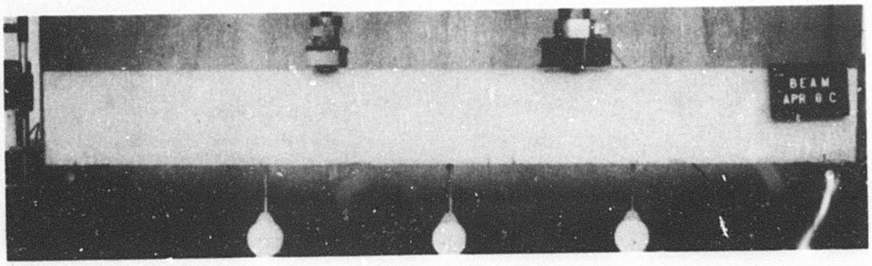


g. Closeup at failure

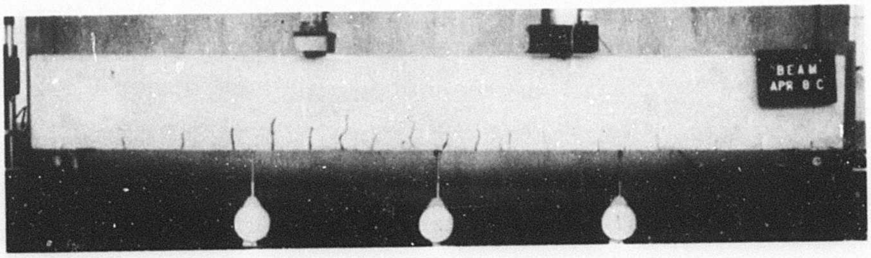
Photograph 10 (2 of 2 sheets)



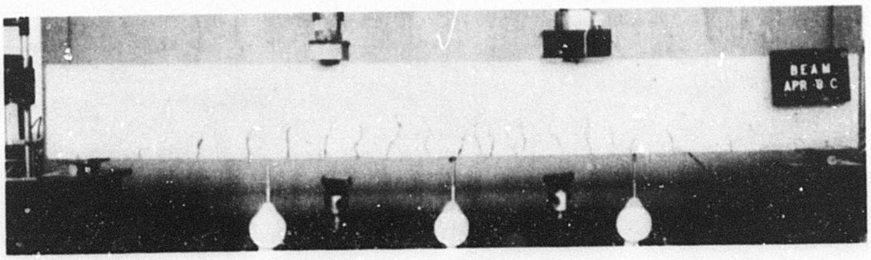
a. No load



b. Total load = 4000 lb (1814.4 kg)

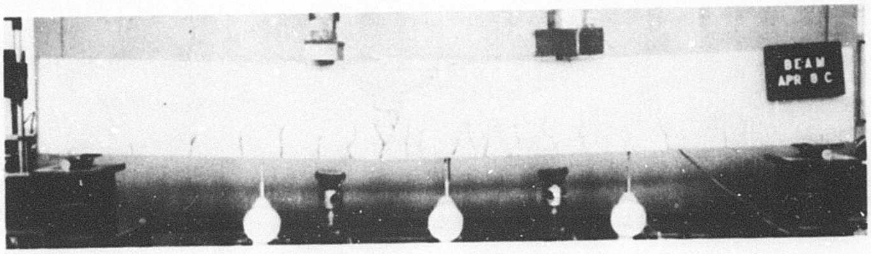


c. Total load = 9000 lb (4082.3 kg)

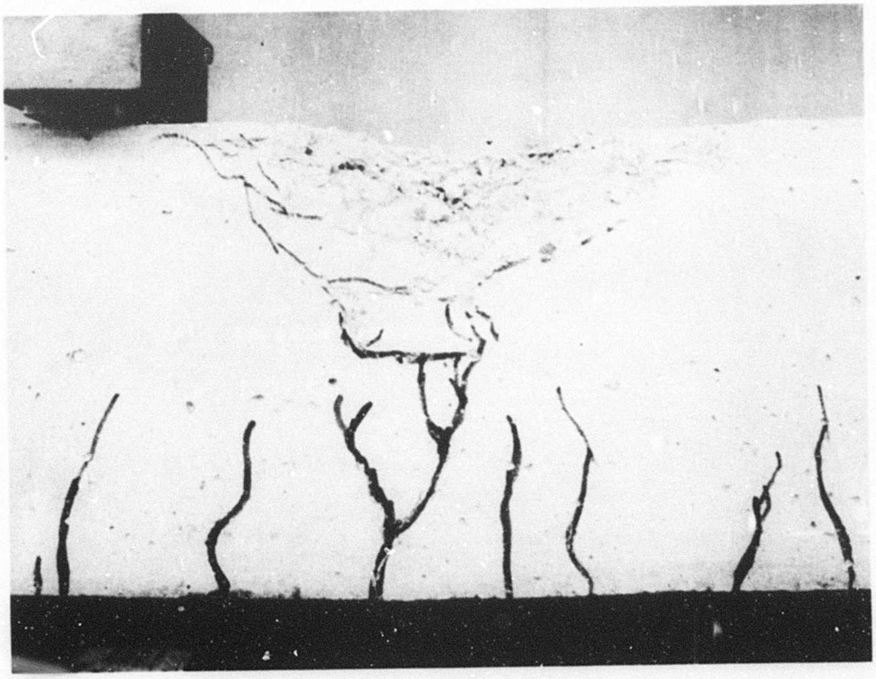


d. Total load = 18,000 lb (8164.7 kg)

Photograph 11. Crack pattern, beam LM2 (1 of 2 sheets)

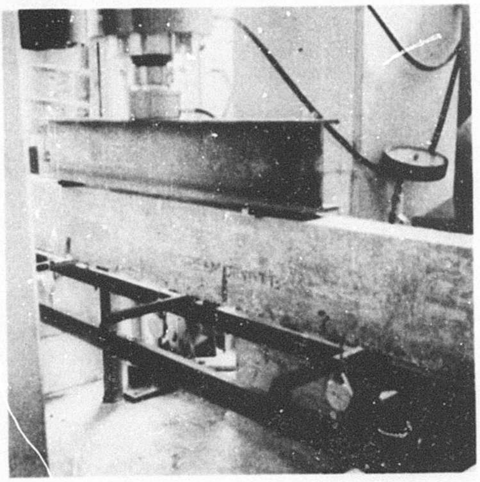


e. Total load = 19,350 lb (8777.0 kg)

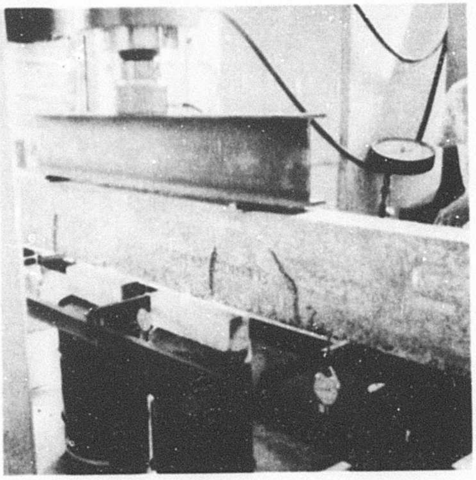


f. Closeup at failure

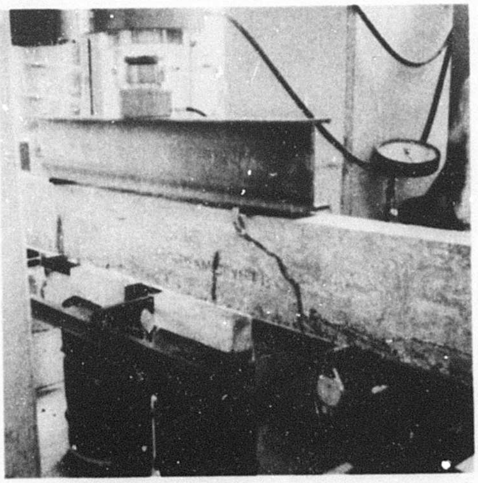
Photograph 11 (2 of 2 sheets)



a. Total load = 3500 lb (1587.6 kg)

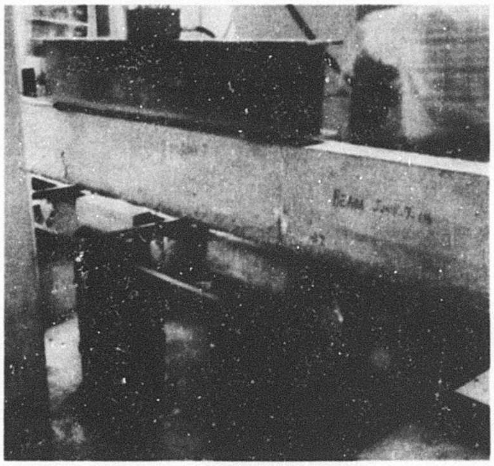


b. Total load = 8500 lb (3855.5 kg)

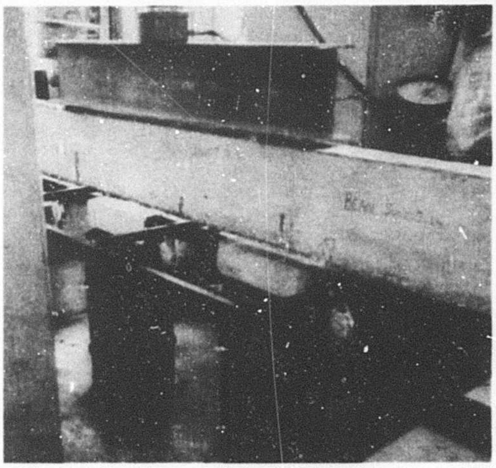


c. Total load = 10,500 lb (4762.7 kg), failure

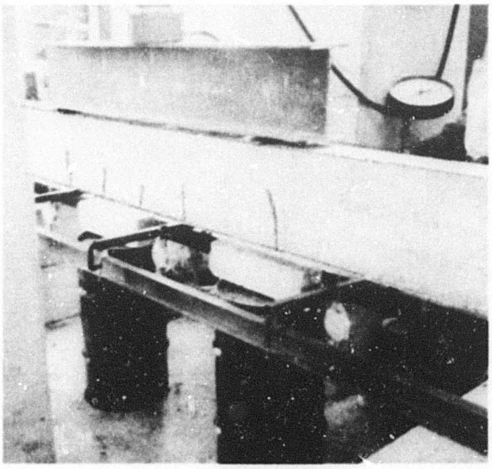
Photograph 12. Crack pattern, beam AM2-1



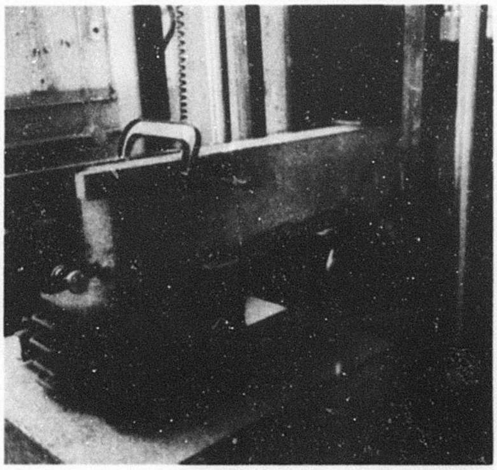
a. Total load = 3000 lb
(1360.8 kg)



b. Total load = 5500 lb
(2494.8 kg)

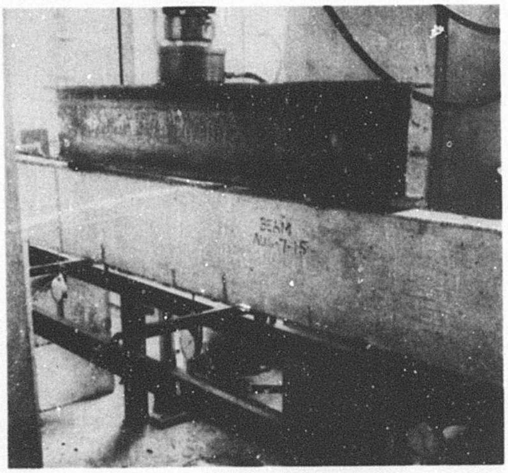


c. Total load = 9000 lb
(4082.3 kg)

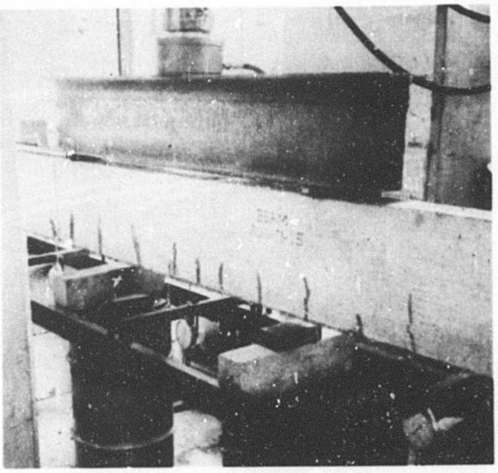


d. Total load = 10,500 lb
(4762.7 kg), failure

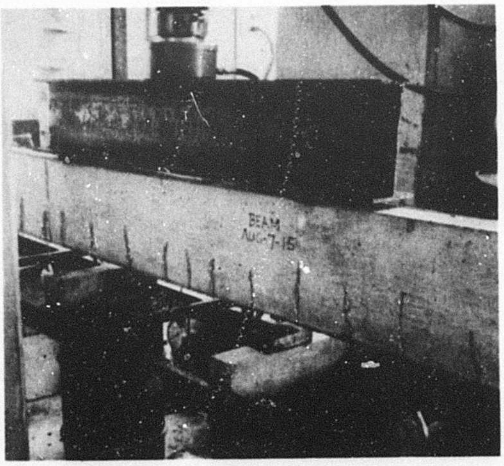
Photograph 13. Crack pattern, beam AM2-2



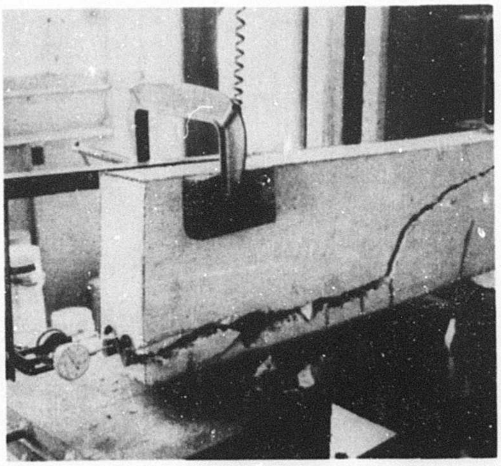
a. Total load = 6500 lb
(2948.4 kg)



b. Total load = 9500 lb
(4309.1 kg)

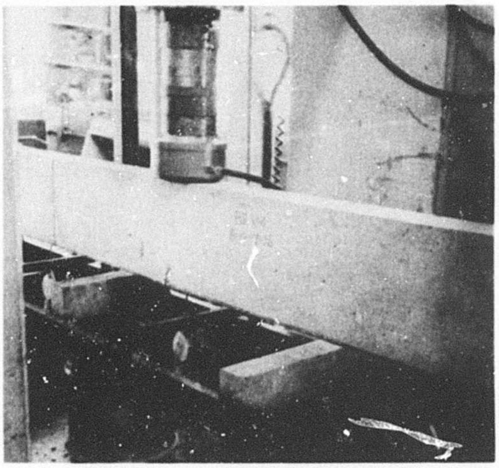


c. Total load = 13,500 lb
(6123.5 kg)

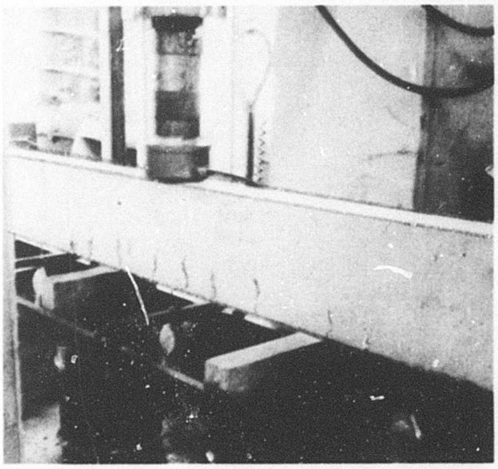


d. Total load = 16,000 lb
(7257.5 kg), failure

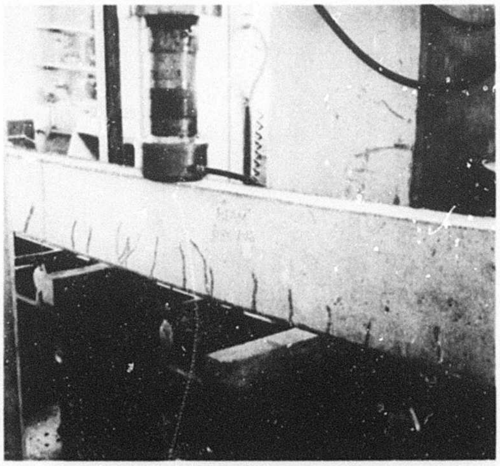
Photograph 14. Crack pattern, beam AM2-3



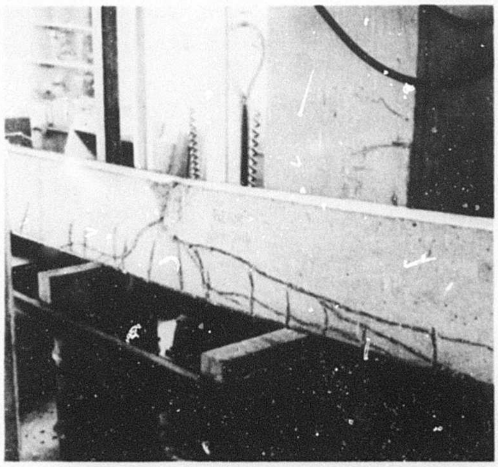
a. Total load = 5500 lb (2494.8 kg)



b. Total load = 11,000 lb (4989.5 kg)

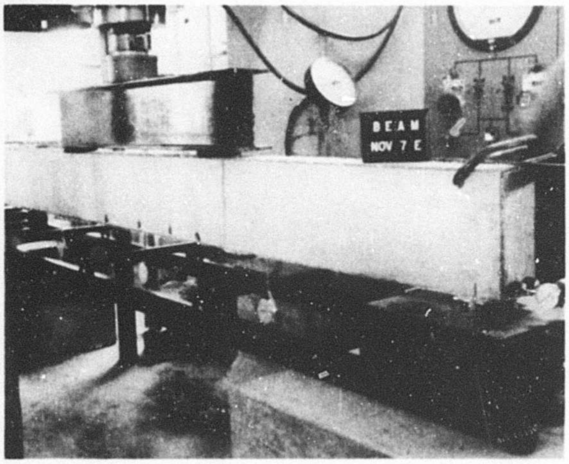


c. Total load = 16,000 lb (7257.5 kg)

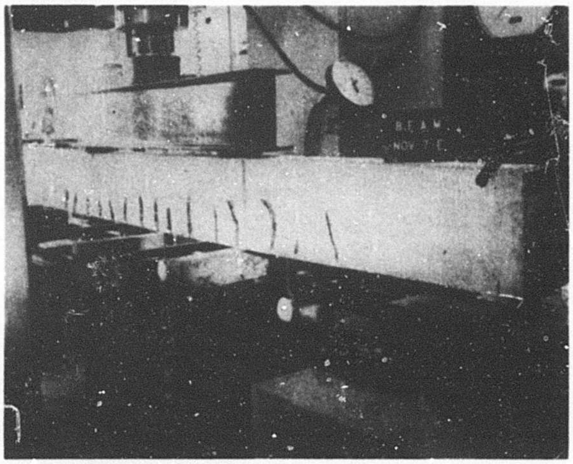


d. 0 load after failure

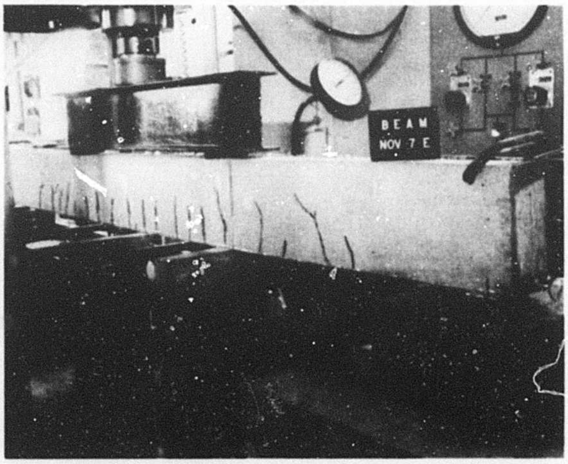
Photograph 15. Crack pattern, beam AM2-4



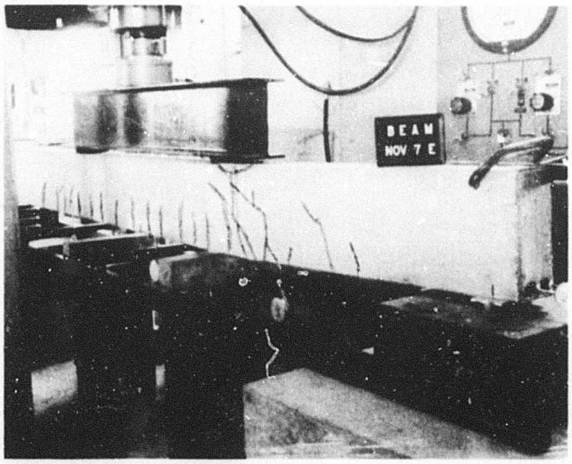
a. Total load = 4000 lb
(1814.4 kg)



b. Total load = 15,000 lb
(6803.9 kg)

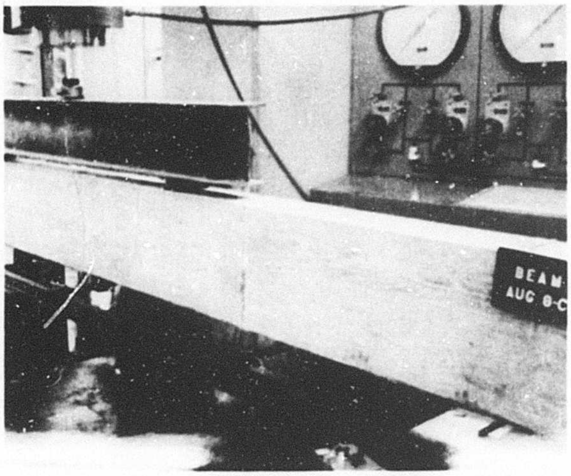


c. Total load = 22,000 lb
(9979.0 kg)

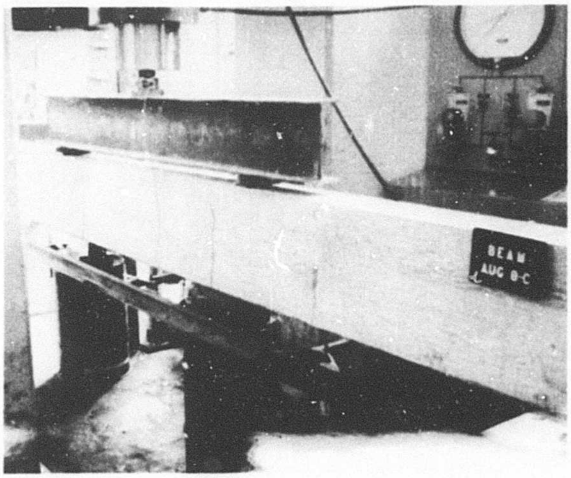


d. Total load = 22,800 lb
(10,341.9 kg), failure

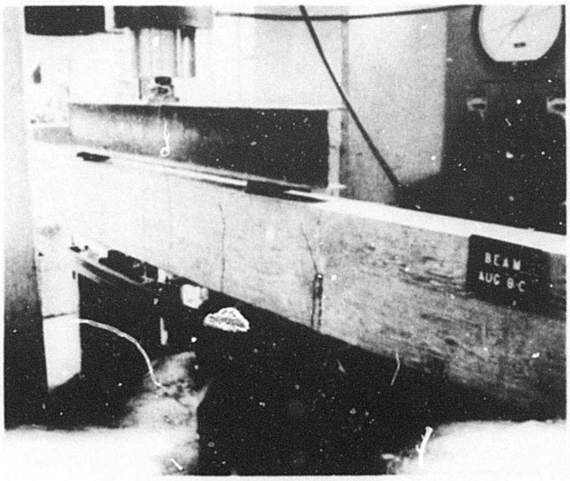
Photograph 16. Crack pattern, beam AM2-5



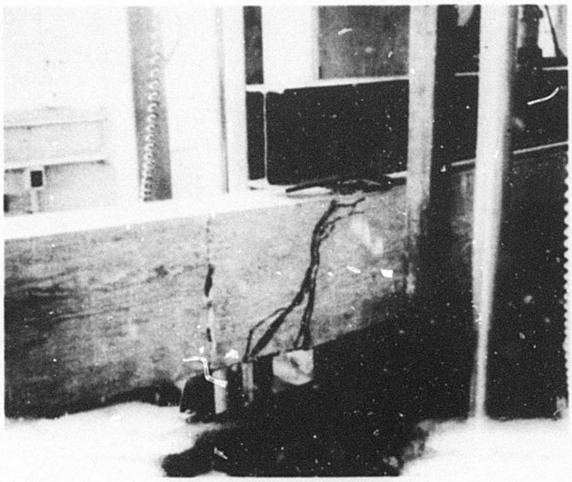
a. No load



b. Total load = 2000 lb
(907.2 kg)



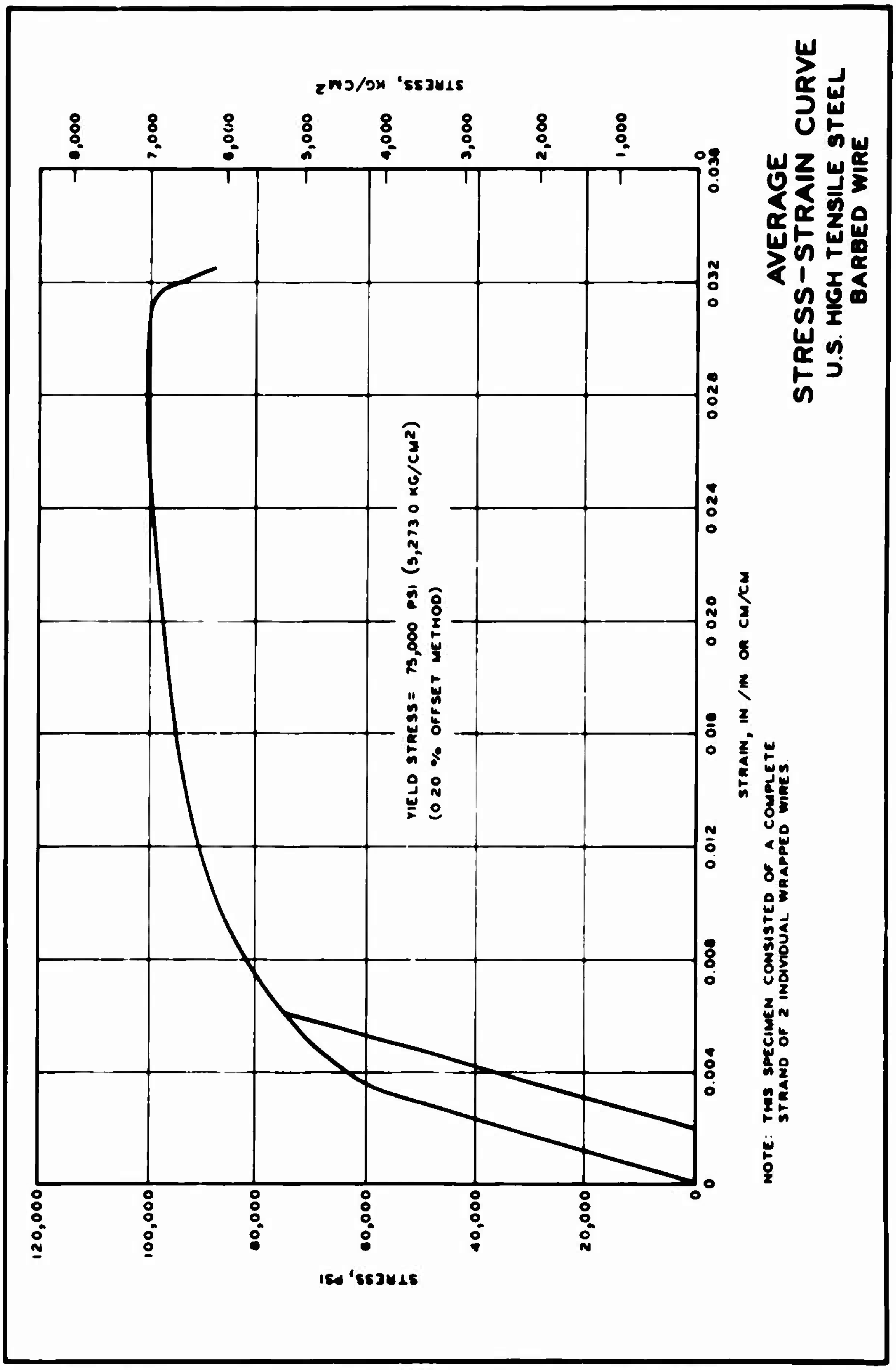
c. Total load = 6000 lb
(2721.6 kg)



d. Total load = 10,000 lb
(4535.9 kg), failure

Photograph 17. Crack pattern, beam WR3

BLANK PAGE



AVERAGE
STRESS-STRAIN CURVE
U.S. HIGH TENSILE STEEL
BARBED WIRE
STRESS, KG/CM²
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
STRESS, PSI
120,000
100,000
80,000
60,000
40,000
20,000
0
0
0.004
0.008
0.012
0.016
0.020
0.024
0.028
0.032
0.036
STRAIN, IN/IN OR CM/CM
YIELD STRESS= 75,000 PSI (5,273.0 KG/CM²)
(0.20 % OFFSET METHOD)
NOTE: THIS SPECIMEN CONSISTED OF A COMPLETE STRAND OF 2 INDIVIDUAL WRAPPED WIRES.

PLATE I

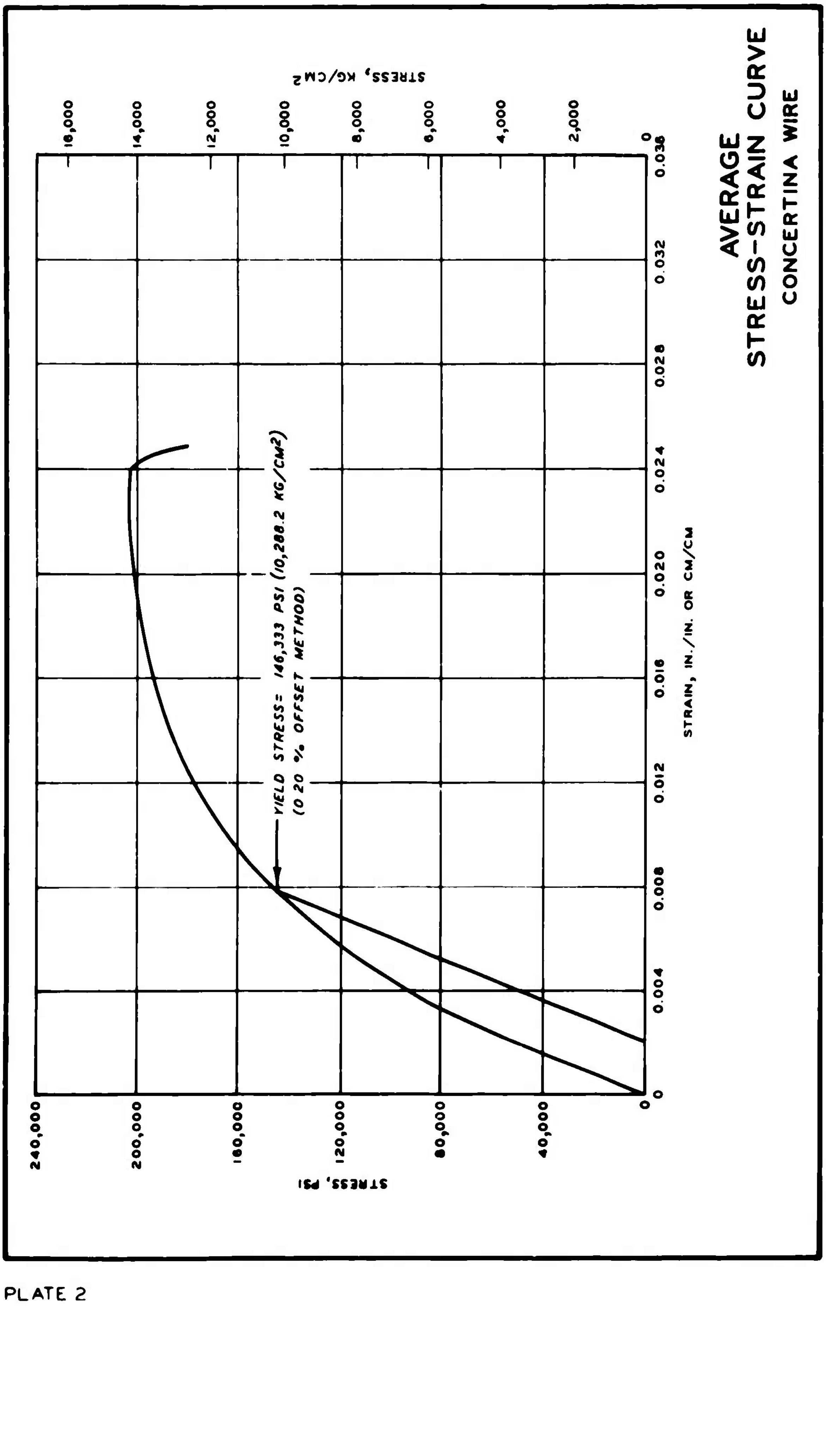
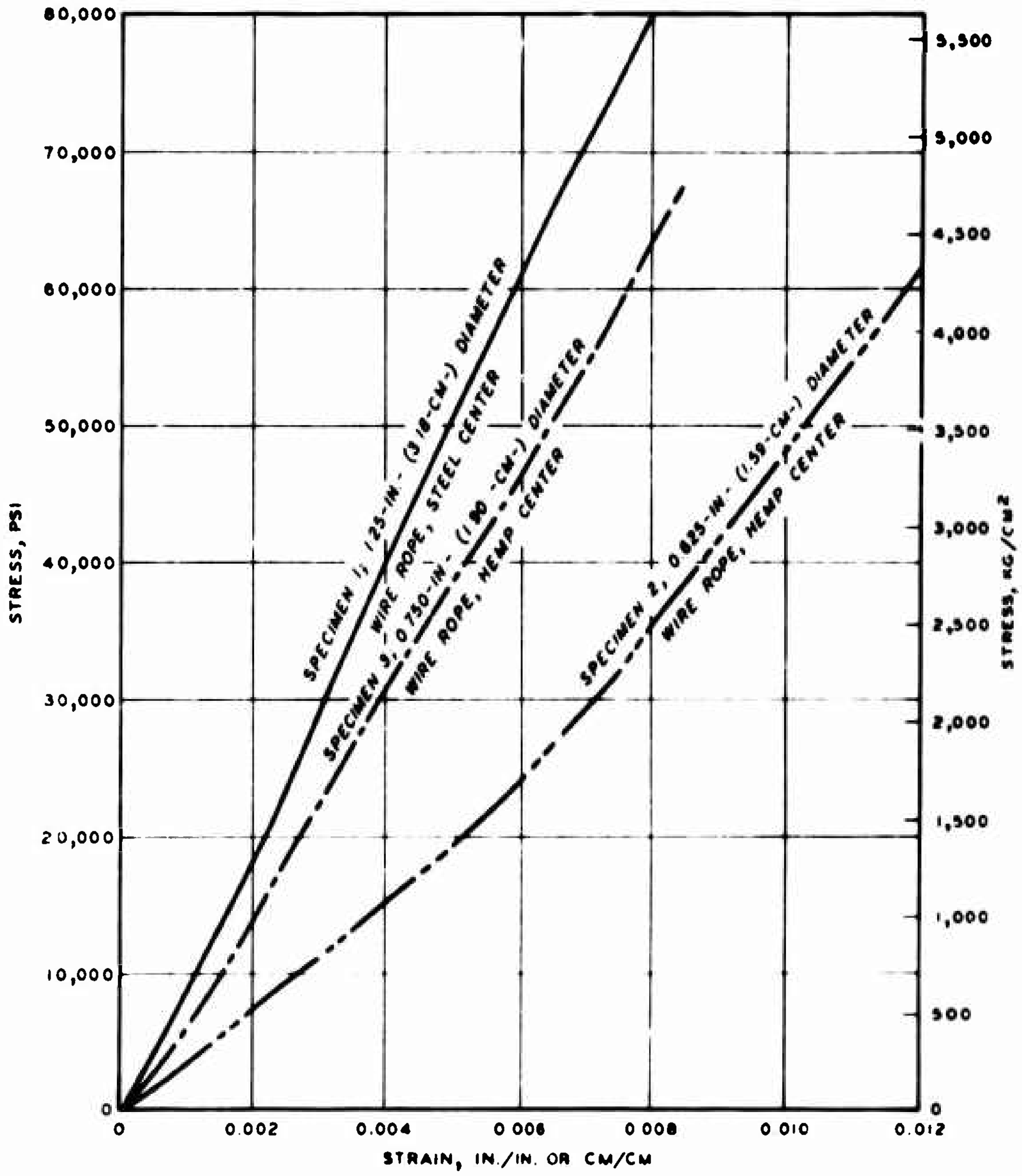
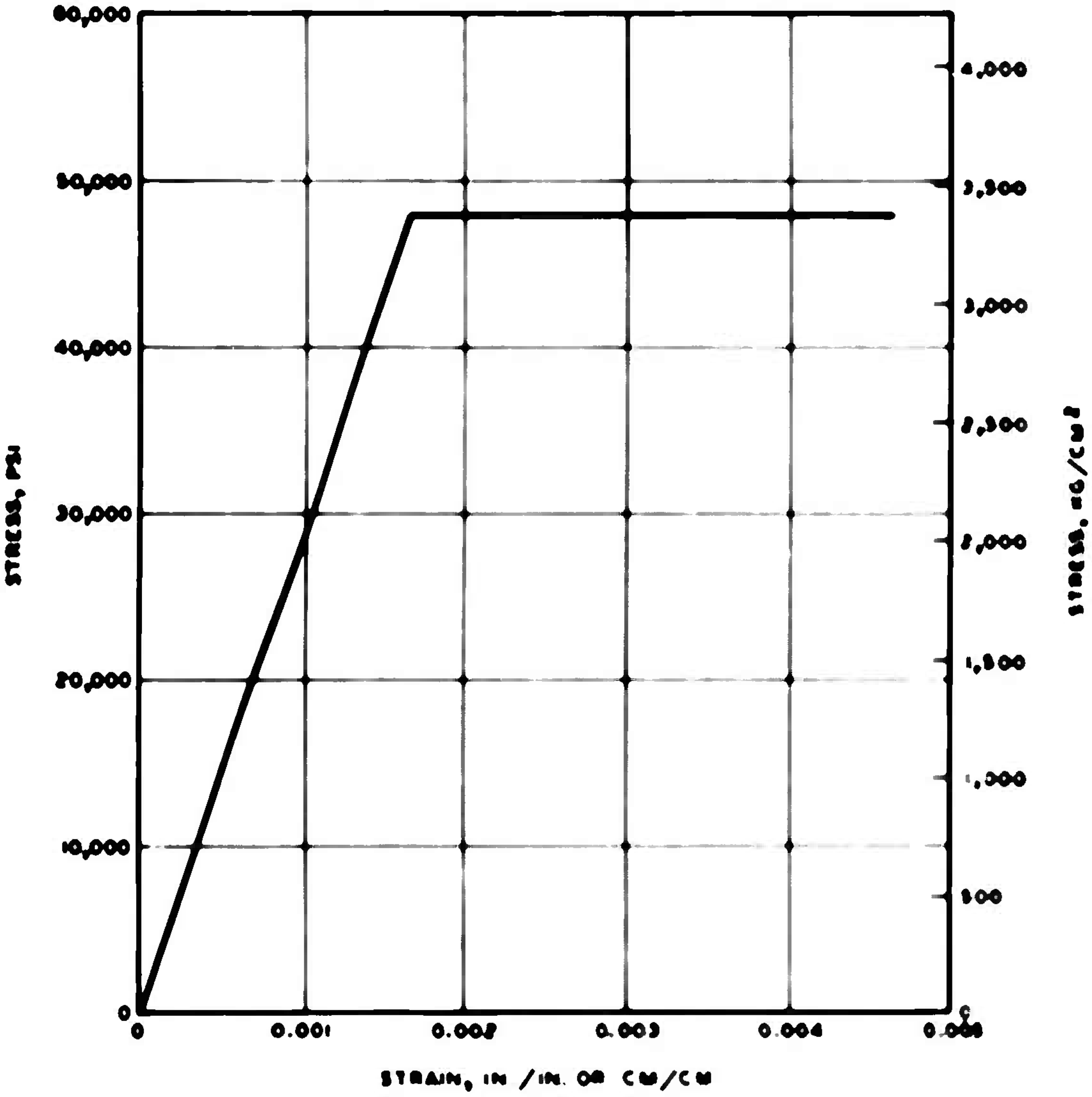

AVERAGE
STRESS-STRAIN CURVE
CONCERTINA WIRE
YIELD STRESS= 146,333 PSI (10,288.2 KG/CM²)
(0.20 % OFFSET METHOD)
STRAIN, IN./IN. OR CM/CM
STRESS, PSI
STRESS, KG/CM²
0
0.004
0.008
0.012
0.016
0.020
0.024
0.028
0.032
0.036
40,000
80,000
120,000
160,000
200,000
240,000
2,000
4,000
6,000
8,000
10,000
12,000
14,000
16,000

PLATE 2



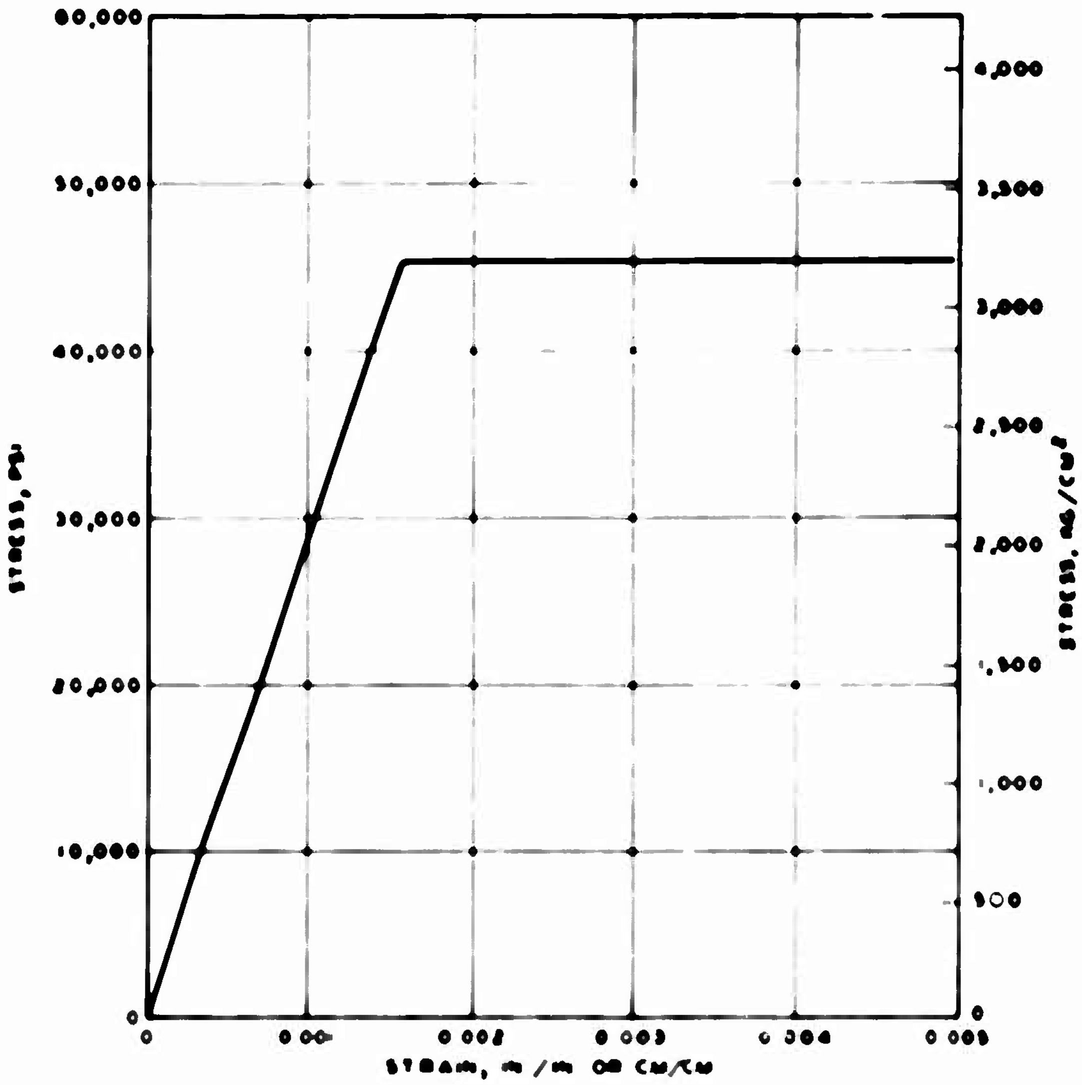
NOTE TO PREVENT INSTRUMENT DAMAGE, SPECIMENS WERE TESTED ONLY TO 75 % OF THEIR ESTIMATED ULTIMATE STRENGTH.

TYPICAL
STRESS-STRAIN CURVES
WIRE ROPE

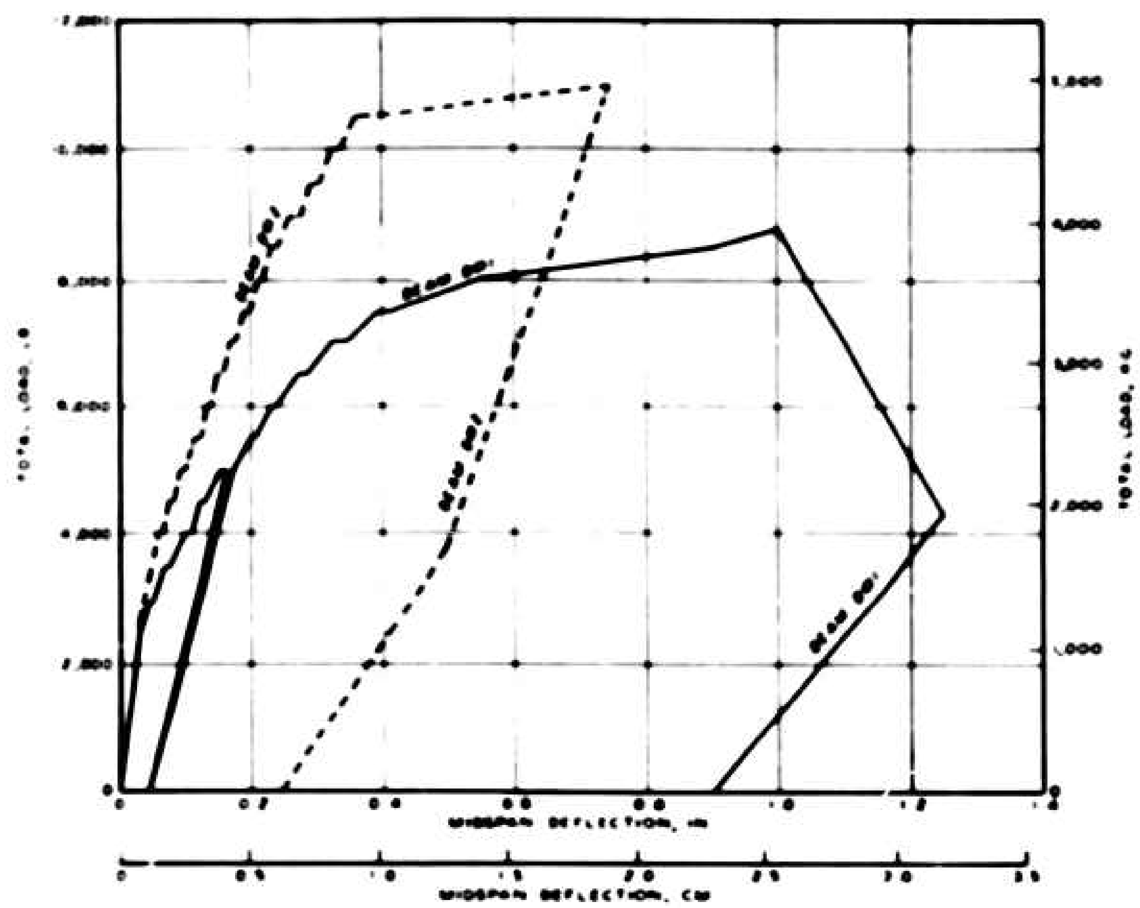


AVERAGE
STRESS-STRAIN CURVE
M-8 PIERCED STEEL
LANDING MAT

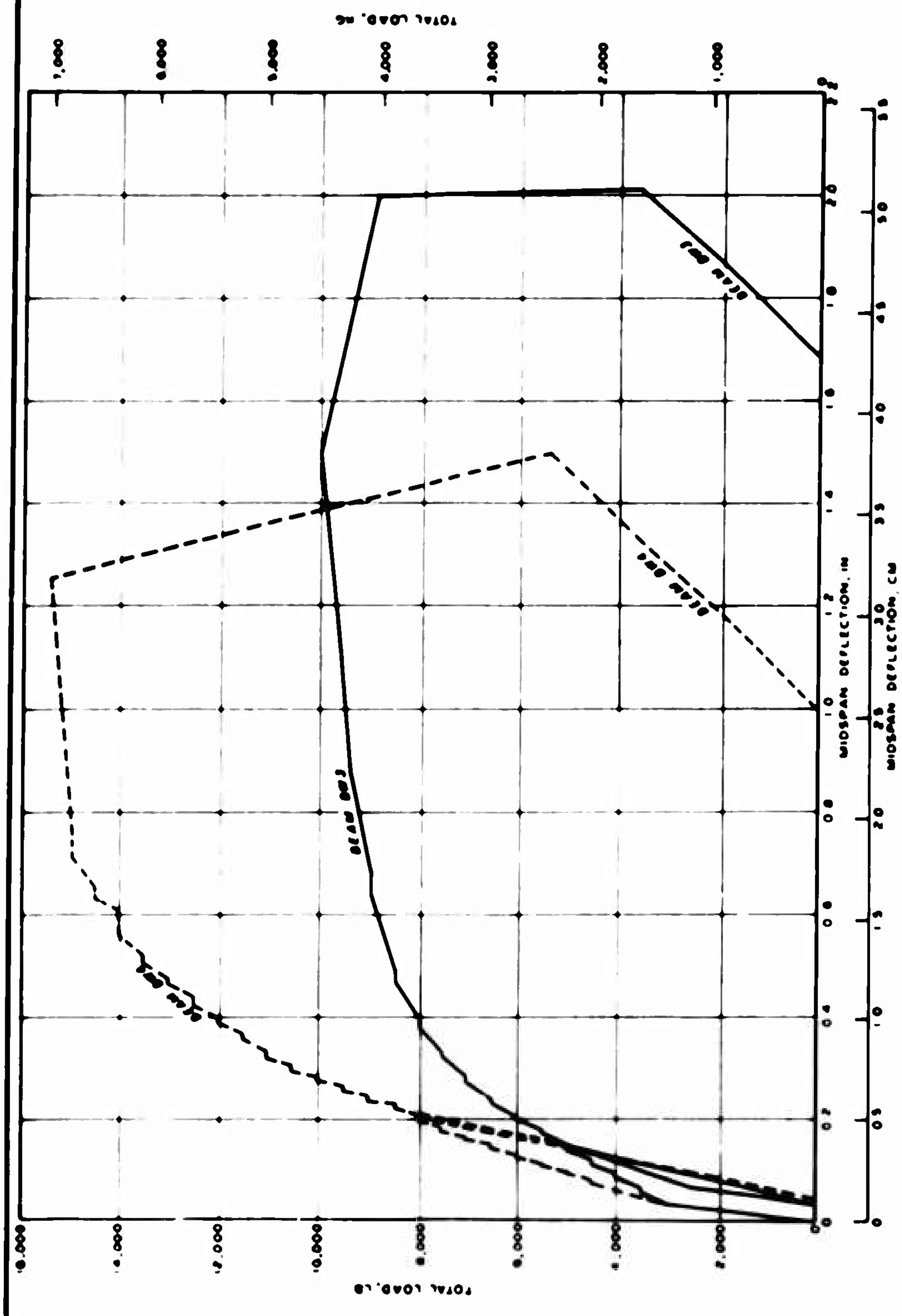
PLATE 4



AVERAGE
STRESS-STRAIN CURVE
AM2 LANDING MAT TIE BARS

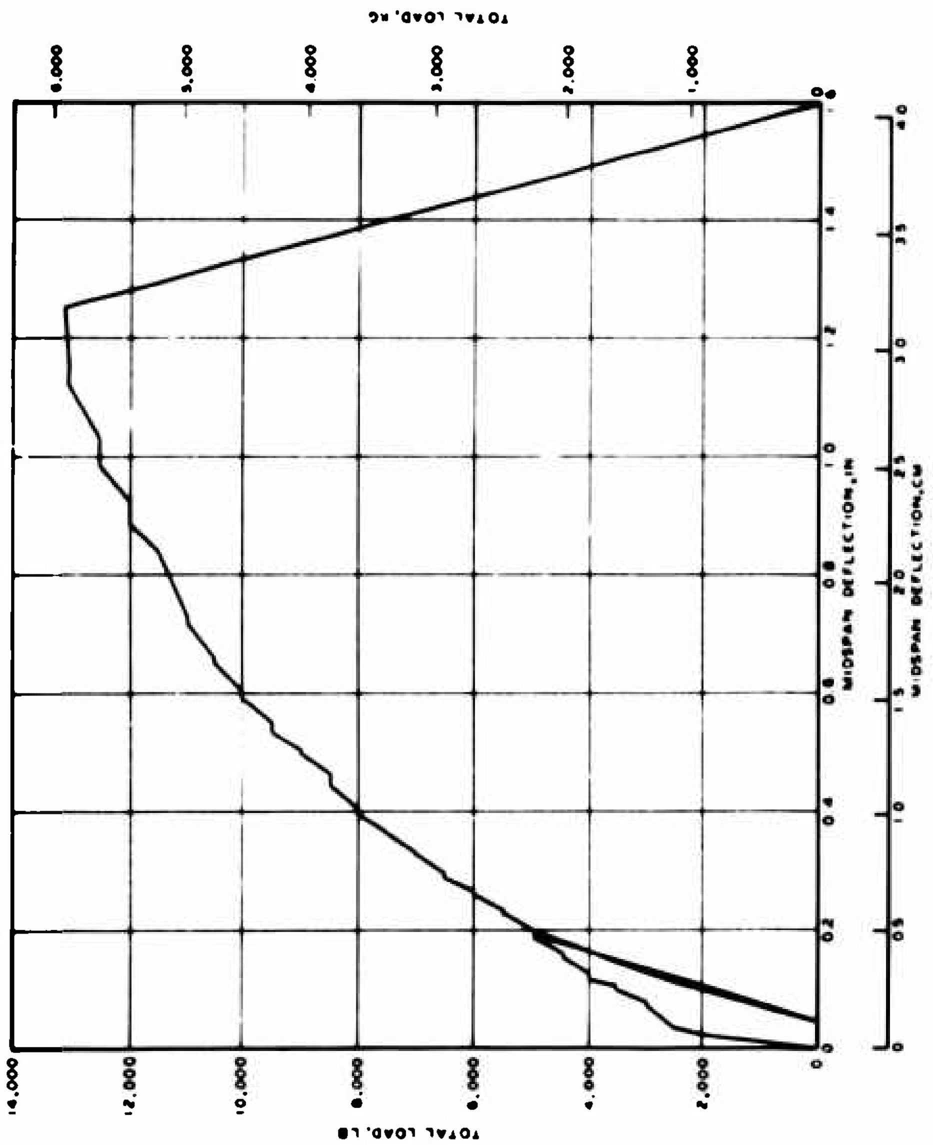


LOAD VS MIDSPAN DEFLECTION
SMALL BEAMS, GROUP I
BEAMS BW1 AND BW2



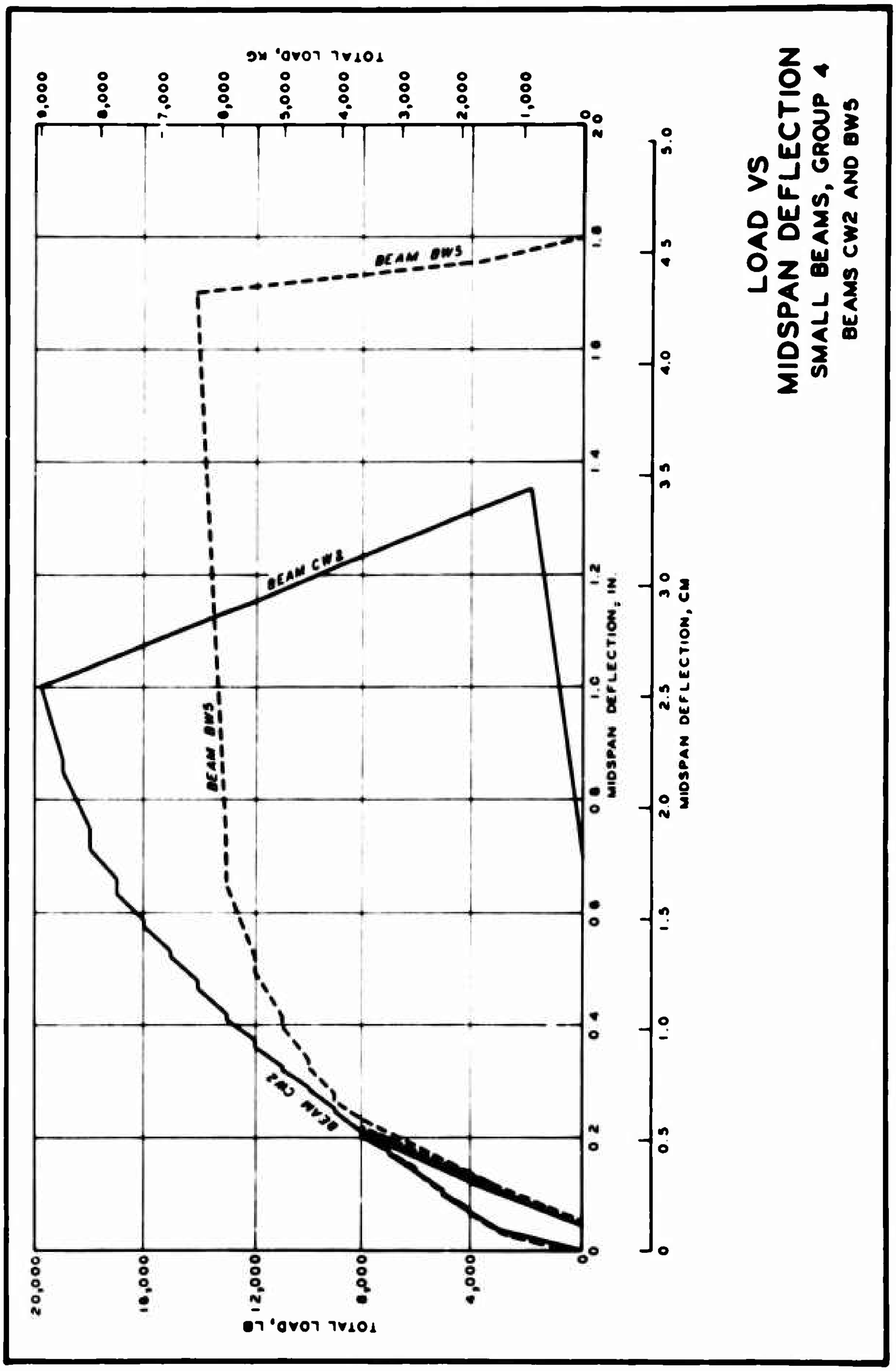
LOAD VS MIDSPAN DEFLECTION
SMALL BEAMS, GROUP 2
BEAMS BW3 AND BW4

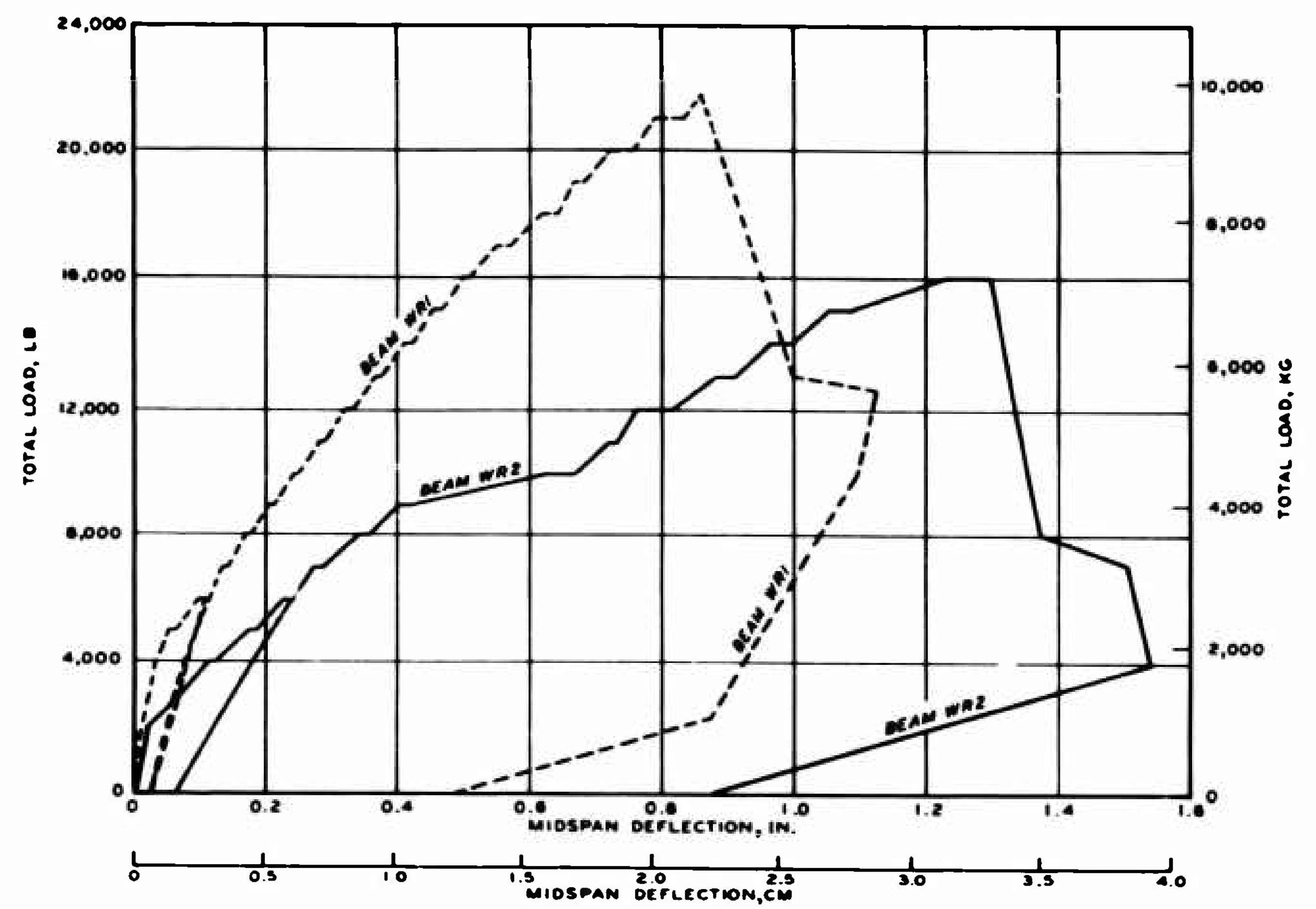
PLATE 7



LOAD VS MIDSPAN DEFLECTION
SMALL BEAMS, GROUP 3
BEAM CW1

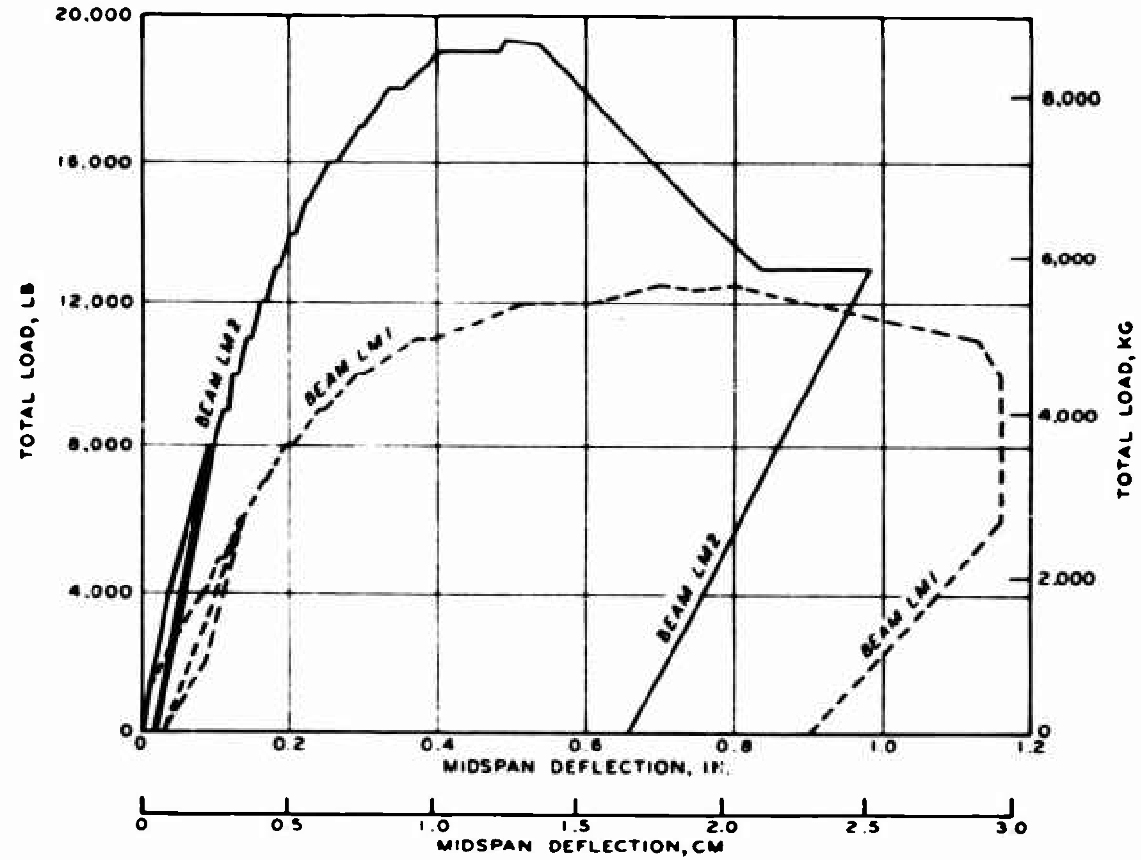
PLATE 8


LOAD VS
MIDSPAN DEFLECTION
SMALL BEAMS, GROUP 4
BEAMS CW2 AND BW5
BEAM BW5
BEAM CW2
BEAM CW2
BEAM BW5
TOTAL LOAD, KG
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
TOTAL LOAD, LB
20,000
16,000
12,000
8,000
4,000
0
MIDSPAN DEFLECTION, IN.
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
MIDSPAN DEFLECTION, CM
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0

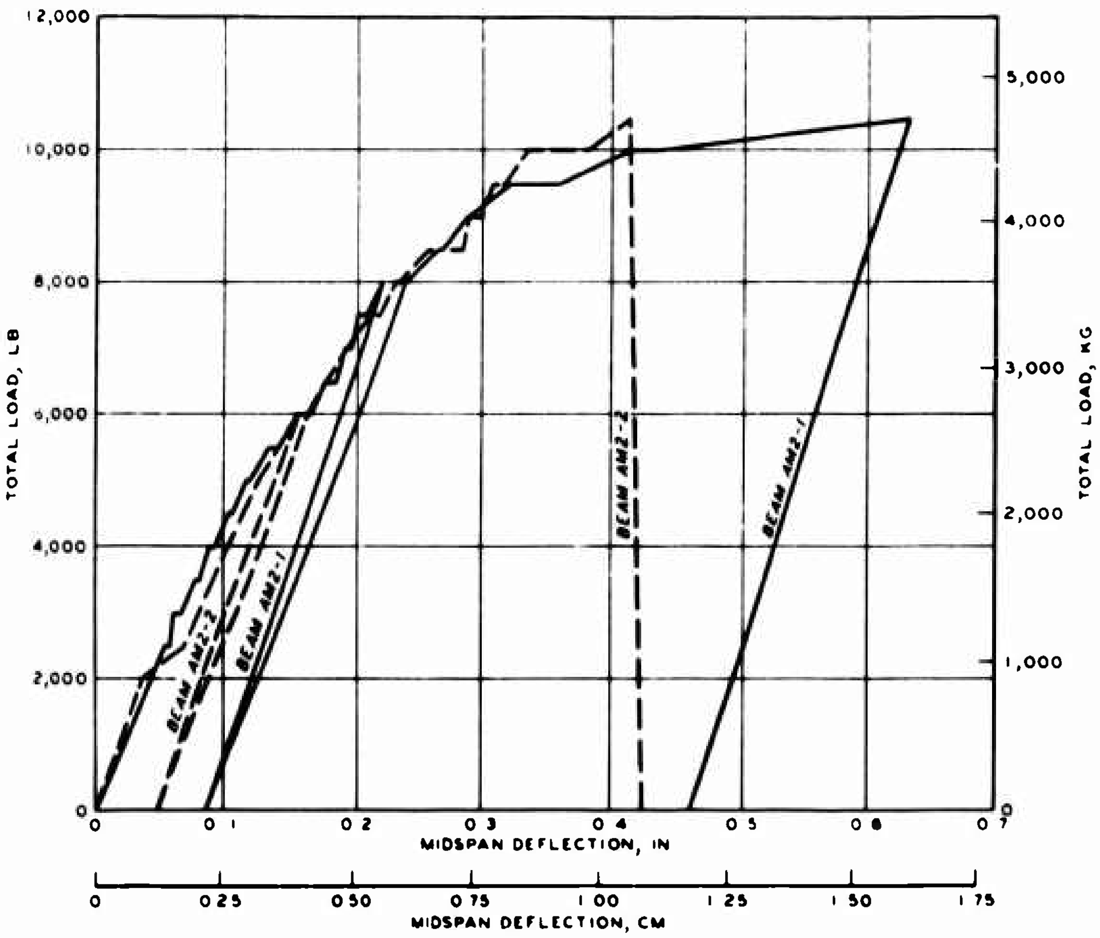


LOAD VS MIDSPAN DEFLECTION
SMALL BEAMS, GROUPS 5 AND 6
BEAMS WR1 AND WR2

PLATE 10

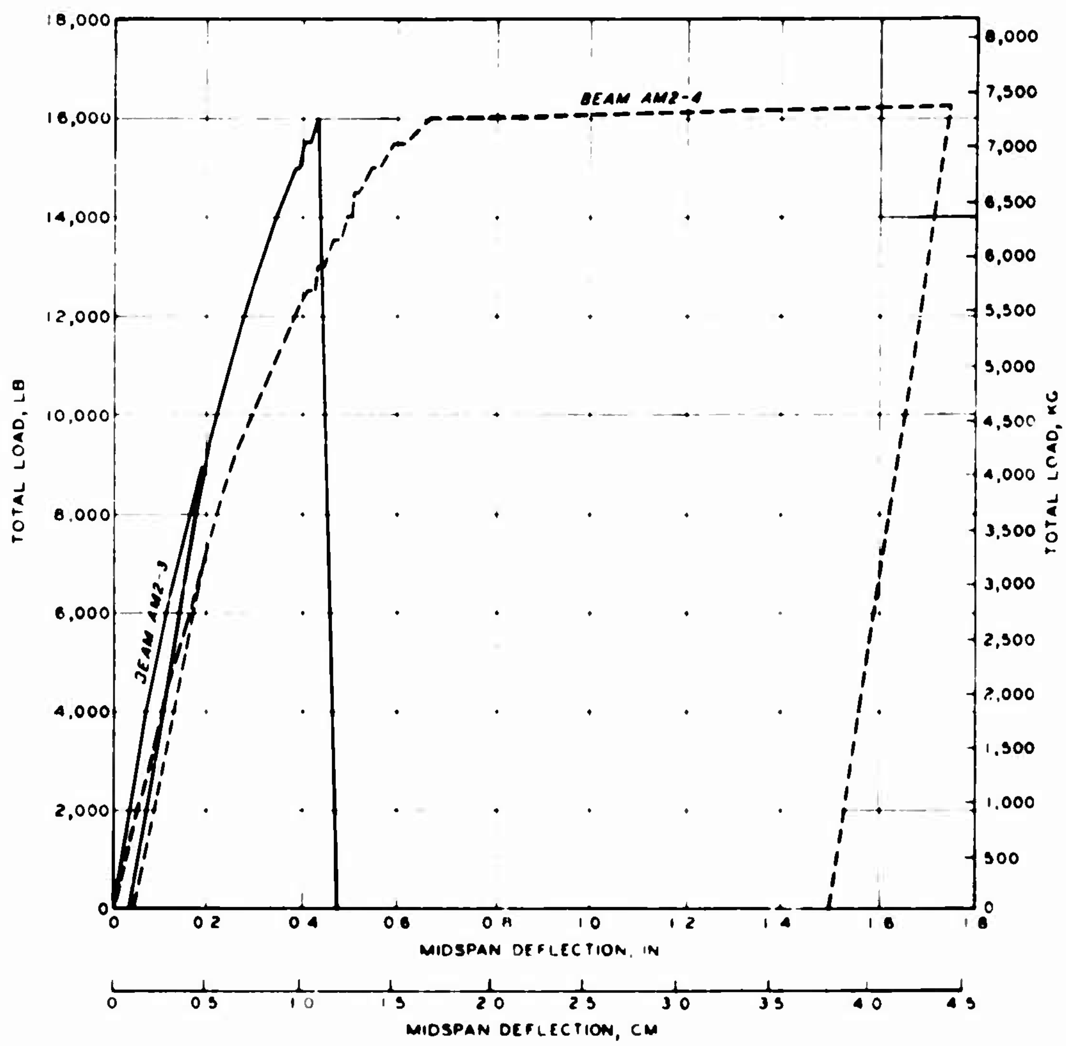


LOAD VS
MIDSPAN DEFLECTION
SMALL BEAMS, GROUPS 6 AND 7
BEAMS LM1 AND LM2



LOAD VS
MIDSPAN DEFLECTION
LARGE BEAMS, GROUP I
BEAMS AM2-1 AND AM2-2

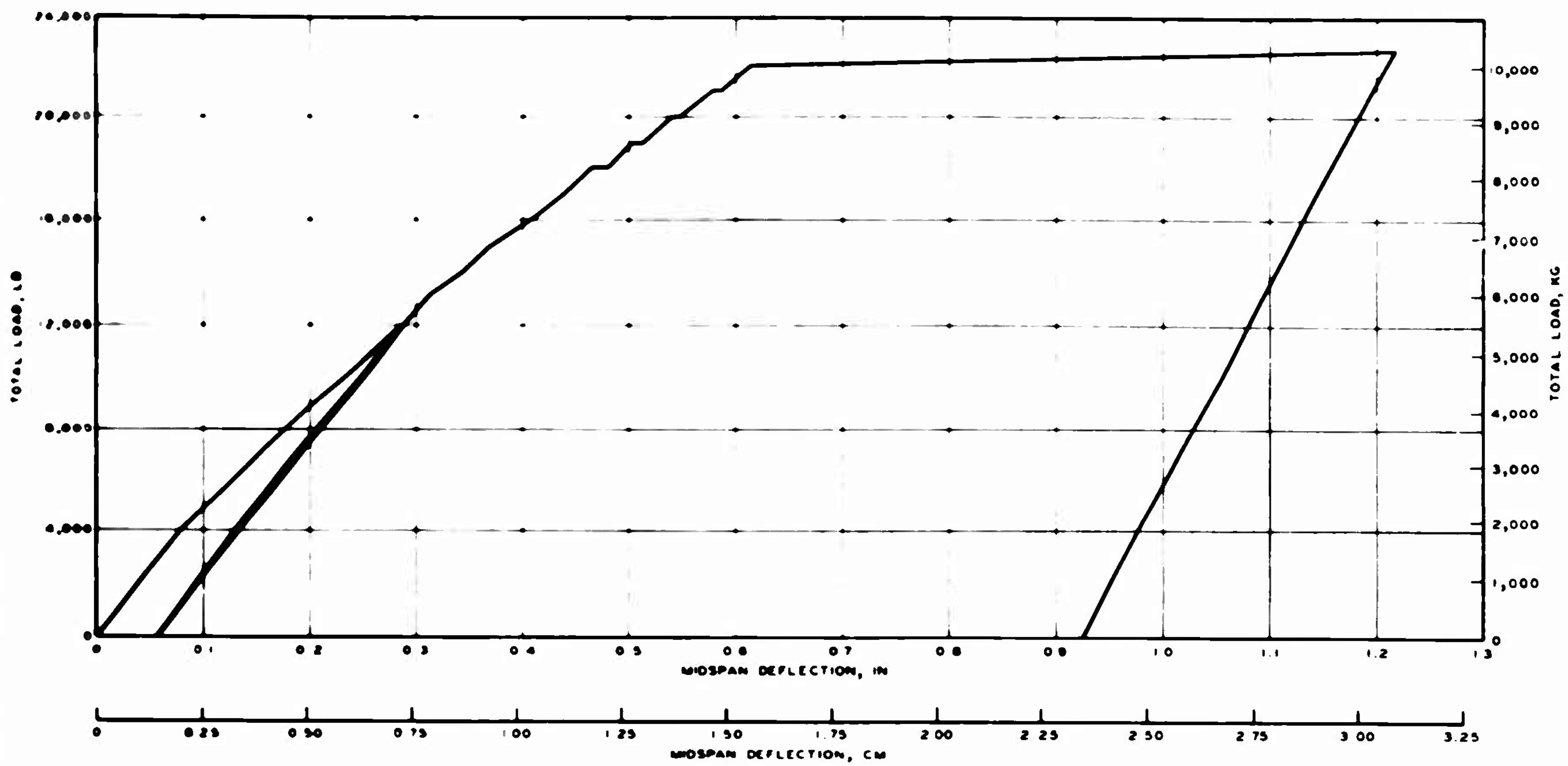
PLATE 12



LOAD VS MIDSPAN DEFLECTION

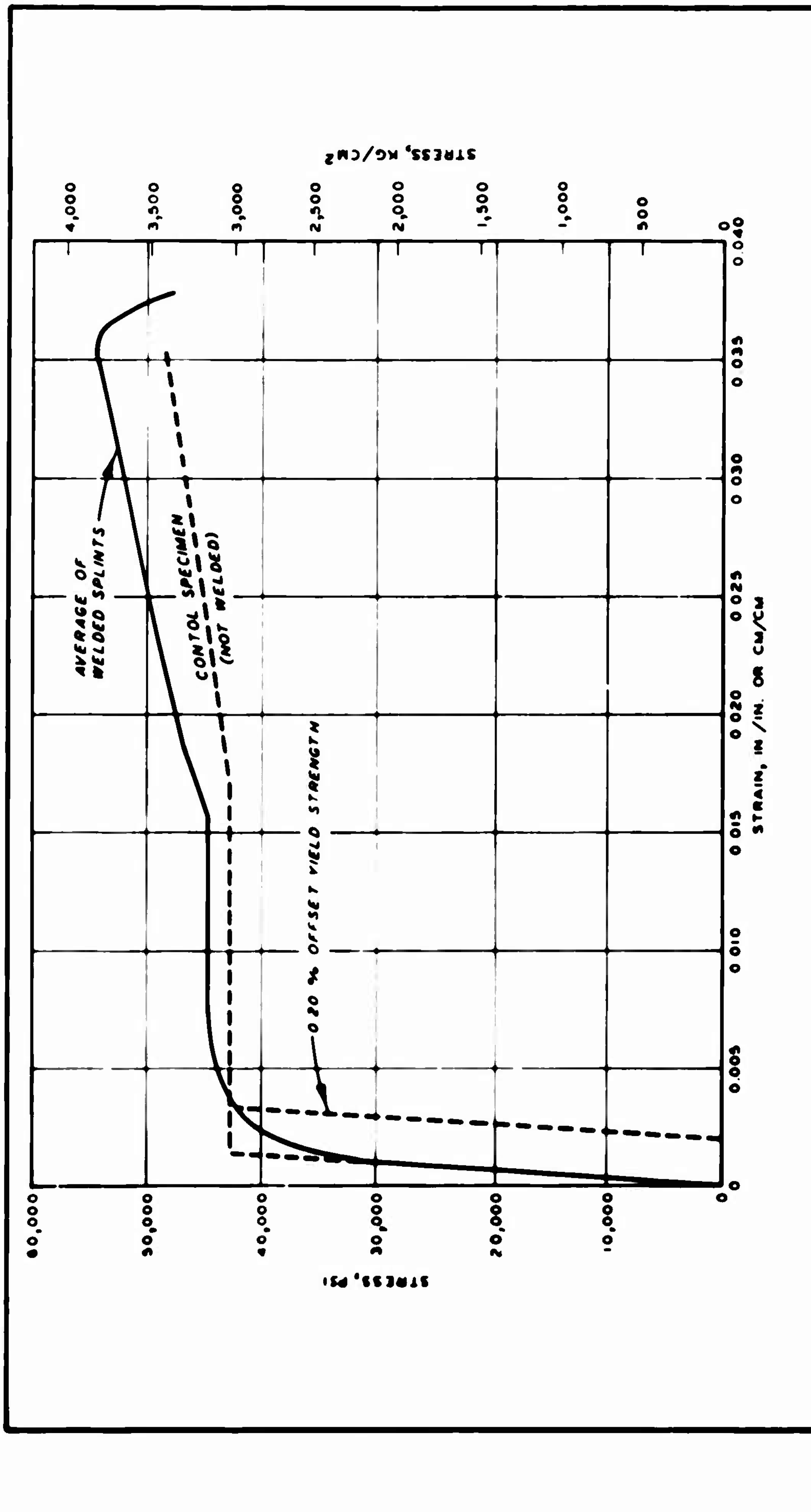
LARGE BEAMS, GROUP 2

BEAMS AM2-3 AND AM2-4

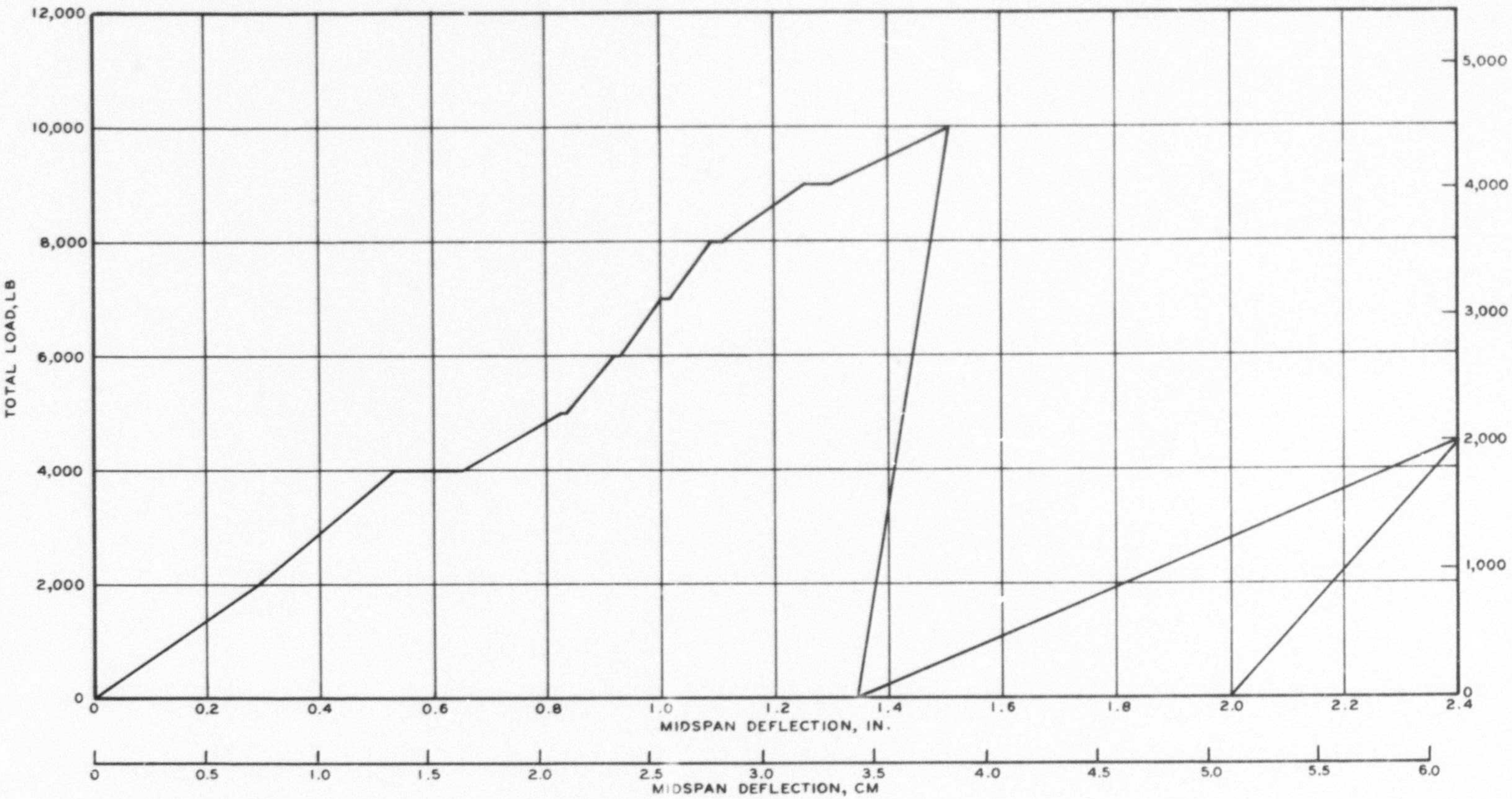


LOAD VS MIDSPAN DEFLECTION
LARGE BEAMS, GROUP 3
BEAM AM2-5

PLATE 14



STRESS-STRAIN CURVE
WELDED AM2 LANDING MAT
TIE BAR SPLINTS



LOAD VS MIDSPAN DEFLECTION
LARGE BEAMS, GROUP 4
BEAM WR3

APPENDIX A: SINHA-FERGUSON ANALYSIS

1. The following is a typical example (beam BW5) illustrating the Sinha-Ferguson method[7] used to analyze the beams which were reinforced with either barbed wire, concertina wire, wire rope, or sections of M8 pierced steel landing mat.

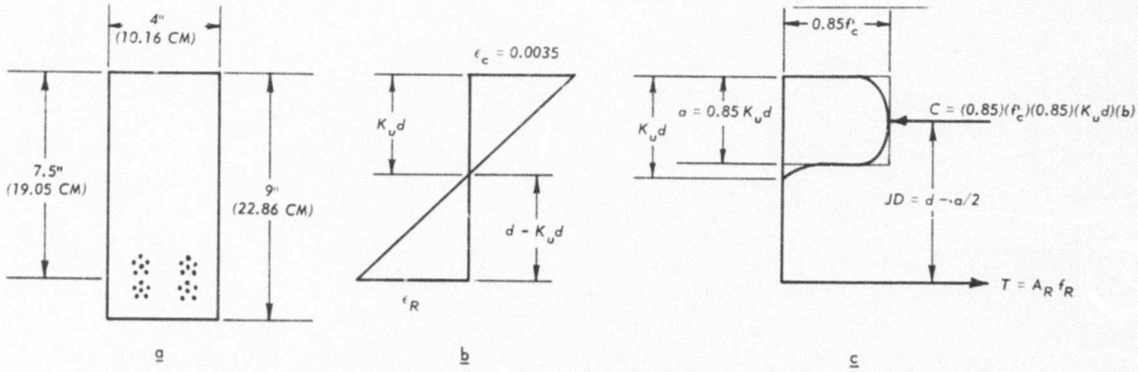


Fig. A1

2. The subsequent computations are based on the assumptions in paragraph 118 of the main text, a concrete compressive strength of 3720 psi (261.5 kg/cm^2), and a reinforcement area of 0.298 in.^{2} (1.922 cm^2).

3. Assume $K_u d$ = 2.25 in. or 5.72 cm.

<u>a</u>. From fig. A1c:

$C = (0.85)(2.25\text{ in.})(0.85)(3720\text{ psi})(4\text{ in.}) = 24{,}189$ lb or 10,971.9 kg.

<u>b</u>. From fig. A1b:

$$\frac{0.0035}{2.25} = \frac{\epsilon_R}{7.50 - 2.25}$$

$$\epsilon_R = 0.00817$$

<u>c</u>. From barbed wire stress versus strain curve (plate 1), f_R = 82,300 psi or 5786.3 kg/cm^2.

<u>d</u>. $T = (A_R)(f_R) = (0.298\text{ in.}^2)(82{,}300\text{ psi}) = 24{,}525$ lb or 11,124.4 kg.

<u>e</u>. $T - C > 200$ lb or 90.72 kg ; therefore, must assume another value for $K_u d$.

4. Assume $K_u d = 2.27$ in. or 5.77 cm.

   a. From fig. A1c:

   $C = (0.85)(2.27 \text{ in.})(0.85)(3720 \text{ psi})(4 \text{ in.}) = 24{,}404$ lb or 11,069.5 kg.

   b. From fig. A1b:

   $$\frac{0.0035}{2.27} = \frac{\epsilon_R}{7.50 - 2.27}$$

   $$\epsilon_R = 0.00806$$

   c. From barbed wire stress versus strain curve (plate 1), $f_R = 82{,}000$ psi or 5765.2 kg/cm^2.

   d. $T = (A_R)(f_R) = (0.298 \text{ in.}^2)(82{,}000 \text{ psi}) = 24{,}436$ lb or 11,084.0 kg.

   e. $T - C = 32$ lb; therefore, assume $T = C$.

   f. $M = C\left(d - \frac{a}{2}\right)$

   $a = 0.85K_u d = (0.85)(2.27 \text{ in.}) = 1.93$ in. or 4.90 cm

   $\frac{a}{2} = \frac{1.93 \text{ in.}}{2} = 0.965$ in. or 2.45 cm

   $M = 24{,}404 \text{ lb } (7.50 \text{ in.} - 0.965 \text{ in.})$

   $M = 159{,}480$ in.-lb or 1837.4 m-kg

1. The following is a typical example (beam AM2-5) illustrating the elastic analysis used to estimate the reinforcement stress at failure.

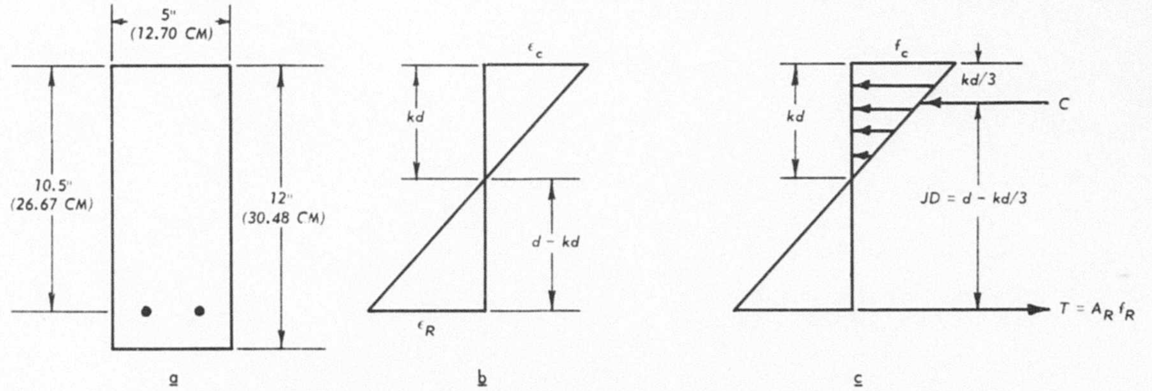


Fig. B1

2. The subsequent computations are based on the following assumptions.

a. f'_c = 4200 psi or 295.3 kg/cm^2.

b. d = 10.50 in. or 26.67 cm.

c. A_R = 1.57 in.^{2} or 10.13 cm^2.

d. Weight of concrete = 145 pcf or 2322.9 kg/m^3.

e. $E_c = W^{1.5}\ 33f'_c$ (from ACI Code 318-63[5]).

f. E_R = 28,150,000 psi or 1,979,142 kg/cm^2 (average of specimens with paint removed).

g. Straight-line strain and stress distribution.

h. Perfect bond between reinforcement and concrete until failure.

3. From fig. B1c:

$$C = \frac{1}{2} f_c bkd = T = A_R f_R$$

$$A_R E_R \epsilon_R = \frac{1}{2} E_c \epsilon_c bkd$$

4. From fig. B1b:

$$\frac{\epsilon_c}{kd} = \frac{\epsilon_R}{d - kd}$$

$$\epsilon_c = \frac{\epsilon_R(kd)}{d - kd}$$

$$\frac{1}{2}\,(3.73 \times 10^6 \text{ psi})\ \epsilon_R(kd)(5 \text{ in.})(kd)$$
$$= (1.57 \text{ in.}^2)(28.15 \times 10^6 \text{ psi})(\epsilon_R)(10.5 - kd)$$

$$9.325\ kd^2 = 464.05 - 44.196\ kd$$

$$kd = 5.07 \text{ in. or } 12.88 \text{ cm}$$

5. From fig. B1c:

$$M = \left(10.5 \text{ in.} - \frac{5.07 \text{ in.}}{3}\right) T$$

$$M = (10.5 \text{ in.} - 1.69 \text{ in.})(1.57 \text{ in.}^2)(f_R)$$

$$M = 501{,}600 \text{ in.-lb (from test of beam)} = 13.832 \text{ in.}^3(f_R)$$

$$f_R = 36{,}264 \text{ psi}$$

Use $f_R = 36{,}260$ psi or $2549.3\ \text{kg/cm}^2$

6. The following is a typical example of the modified ultimate strength analysis used to estimate the beam's (beam AM2-5) reinforcement stress at failure.

7. The following computations are valid providing the assumptions are made.

<u>a</u>. $f'_c = 4200$ psi or $295.3\ \text{kg/cm}^2$.

<u>b</u>. $d = 10.50$ in. or 26.67 cm.

<u>c</u>. $A_R = 1.570\ \text{in.}^2$ or $10.126\ \text{cm}^2$.

<u>d</u>. Straight-line distribution of stresses until the effective bond strength of the reinforcement is reached and then continuous slippage between the reinforcement and concrete at a constant tensile strength resulting in a bilinear resistance function equivalent to the bilinear stress-strain curve assumed in ACI Code 318-63[5] (fig. B2).

<u>e</u>. Validity of standard assumptions for ultimate strength design, e.g., straight-line strain distribution, Whitney stress block, ultimate concrete strain of 0.003.

<u>f</u>. Due to the assumptions <u>d</u> and <u>e</u>, equation 16-1 of the ACI Code 318-63[5] may be used by letting $\phi = 1$.

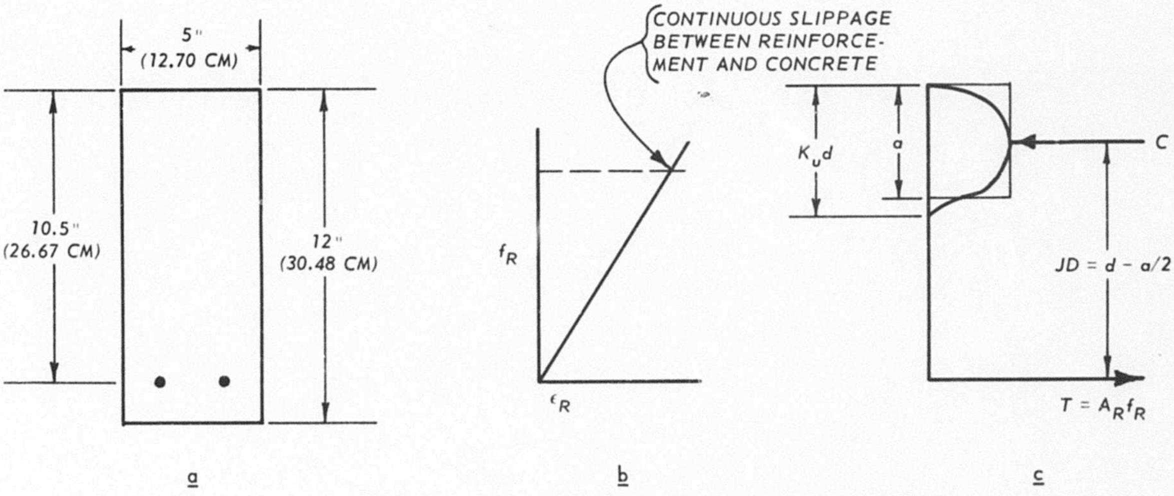


Fig. B2

8. From fig. B2c:

$$M = C(JD) = 0.85f'_c\ 0.85K_u db\left(d - \frac{a}{2}\right)$$

M = 501,600 in.-lb (from test of beam)

$$M = 501,600 \text{ in.-lb}$$
$$= (0.85)(4200 \text{ psi})(0.85)(K_u d)(5 \text{ in.})\left(10.5 \text{ in.} - \frac{a}{2}\right)$$

$$M = 501,600 \text{ in.-lb}$$
$$= (0.85)(4200 \text{ psi})(0.85)(K_u d)(5 \text{ in.})(10.5 \text{ in.} - 0.425K_u d)$$

$$501,600 = 159,311.25K_u d - 6448.31K_u d^2$$

$$K_u d^2 - 24.71K_u d + 77.79 = 0$$

$$K_u d = 3.70 \text{ in. or } 9.398 \text{ cm}$$

$$M = A_R f_R\left(10.5 - \frac{a}{2}\right) = A_R f_R(10.5 - 0.425K_u d)$$

$$501,600 \text{ in.-lb} = 1.57 \text{ in.}^2\ f_R(10.5 \text{ in.} - 1.572 \text{ in.})$$

$$f_R = \frac{501,600 \text{ in.-lb}}{1.57 \text{ in.}^2(8.928 \text{ in.})} = 35,785 \text{ psi}$$

Use 35,790 psi or 2516.3 kg/cm^2

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11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY Office, Chief of Engineers, U. S. Army Washington, D. C.

13. ABSTRACT This is the second of a series of reports on expedient reinforcement for use in Southeast Asia. This report concerns materials generally available near theater of operations areas, specifically, barbed and concertina wire, wire rope, landing mat, and landing mat tie bars. The most important engineering properties (yield strength, tensile strength, elastic modulus, and bond) of the materials were determined and 17 concrete beams were cast and tested to determine the suitability of the materials as reinforcement and to develop design procedures. Each of the materials tested can be used as expedient reinforcement, but due to its method of fabrication, wire rope is the least desirable. Wire rope larger than 3/4 in. (1.90 cm) in diameter is not recommended as expedient reinforcement. Paint should be removed from AM2 landing mat tie bars to make them suitable as reinforcement. If it is necessary to join the tie bars to obtain a sufficient length of reinforcement, the bars should be joined by welding rather than by bolting. Barbed and concertina wire should be placed in assemblies of approximately six strands each to reduce fabrication time. At present shear reinforcement is recommended for all types of reinforcement tested, although the shape of the M8 landing mat tested provides effective partial shear reinforcement when the sections are placed in an upright position. Either barbed or concertina wire stirrups were found to provide sufficient expedient shear reinforcement. A modified ultimate strength (Sinha-Ferguson) method is recommended for the design of beams reinforced with barbed wire, concertina wire, wire rope, or landing mat. Either working stress or ultimate strength design according to ACI Code 318-63 is recommended for tie bar reinforcement. Shear reinforcement can be provided according to ACI Code procedures.

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14. KEY WORDS	LINK A		LINK B		LINK C	
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Landing mats						
Reinforced concrete						
Southeast Asia						
Tie bars						
Wire						
Wire rope						